California High-Speed Train Project

TECHNICAL MEMORANDUM

Track-Structure Interaction
TM 2.10.10

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for the California High-Speed Rail Authority

System Level Technical and Integration Reviews

The purpose of the review is to ensure:

- Technical consistency and appropriateness
- Check for integration issues and conflicts

System level reviews are required for all technical memoranda. Technical Leads for each subsystem are responsible for completing the reviews in a timely manner and identifying appropriate senior staff to perform the review. Exemption to the system level technical and integration review by any subsystem must be approved by the Engineering Manager.

System Level Technical Reviews by Subsystem:

Systems:	Not Required Bradley Banks, PE	Date
Infrastructure:	*Signed document on file* Gene Lusherovich, PE	05 Nov 2013 Date
Operations & Maintenance:	*Signed document on file* Joseph Metzler	25 Oct 2013 Date
Rolling Stock:	*Signed document on file* Frank Banko	04 Oct 2013 Date

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ABSTRACT

This technical memorandum establishes California High-Speed Train Project (CHSTP) requirements for track-structure interaction (TSI) for bridges, aerial structures, grade separations, culverts, and aerial stations supporting high-speed train (HST) tracks. These structures, which are critical to ensuring elevated track performance, are referred to as “TSI-critical structures”.

TSI-critical structures are subject to the following design requirements: structural frequency recommendations, track serviceability limits, rail-structure interaction (RSI) limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction (VTSI) analysis limits.

These requirements are concerned with limiting deformations and accelerations of TSI-critical structures, since the structure response can be dynamically magnified under high-speed moving trains. Excessive deformations and accelerations can lead to unacceptable changes in vertical and horizontal track geometry, excessive rail stress, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

The CHSTP is the first high-speed train system in the United States with design speeds over 200 mph. The Federal Railway Administration (FRA) has published proposed rules [5] addressing vehicle-track interaction safety standards for Class 9 (≥160 mph) track. This technical memorandum uses these proposed rules as a basis for allowable structural deformations during track-structure interaction.

The information included in this document shall be used in conjunction with the following technical memoranda:

- TM 2.1.5: Track Design
- TM 2.3.2: Structure Design Loads
- TM 2.10.4: Seismic Design Criteria

The scope of this technical memorandum is limited to TSI-critical structures with continuous welded rail having no rail expansion joints. Specific criteria is developed for both non-ballasted and ballasted track forms. This scope of this technical memorandum is not intended for tracks supported at grade or upon embankments. Where applicable for rail-structure interaction analyses, this technical memorandum provides guidance for modeling at-grade earthen embankments or cuts at bridge approaches and abutments.

1.0 INTRODUCTION

1.1 PURPOSE OF TECHNICAL MEMORANDUM

This Technical Memorandum (TM) establishes California High-Speed Train Project (CHSTP) requirements for track-structure interaction (TSI) for bridges, aerial structures, grade separations, culverts, and aerial stations supporting high-speed train (HST) tracks.

These requirements encompass structures supporting non-ballasted and ballasted track forms.

This Technical Memorandum shall be used in conjunction with the following Technical Memoranda:

- TM 2.1.5: Track Design
- TM 2.3.2: Structure Design Loads
- TM 2.10.4: Seismic Design Criteria

The scope of this technical memorandum is limited to structures with continuous welded rail (CWR) having no rail expansion joints. This scope of this technical memorandum is not intended for tracks supported at grade or upon embankments.

1.2 STATEMENT OF TECHNICAL ISSUE

This TM establishes track-structure interaction criteria and guidance for new structures supporting high-speed train tracks.

Successful high-speed track performance shall be maintained under service loads with continued operability after a low-level seismic event. These performance requirements are unique to infrastructure designed to support high-speed train operation and therefore may be critical for design.

Structural deformation limitations and dynamic response thresholds based on track performance are developed and implemented as part of the design criteria. These criteria consist of structural frequency recommendations, track serviceability limits, rail-structure interaction (RSI) limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction (VTSI) analysis limits.

1.3 GENERAL INFORMATION

1.3.1 Definition of Terms

The following technical terms and acronyms and abbreviations used in this document have specific connotations with regard to the California High-Speed Train system.

Acronyms/Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
Authority	California High-Speed Rail Authority
CBDS	Caltrans Bridge Design Specifications
CF	Centrifugal Force
CHST	California High-Speed Train
CHSTP	California High-Speed Train Project
CSDC	Caltrans Seismic Design Criteria
CWR	Continuous Welded Rail
EN	EuroCode
FRA	Federal Railway Administration
HST	High-Speed Train
I	Vertical impact effect
I_{LLV}	Vertical impact effect on actual high-speed trains

OBE	Operating Basis Earthquake
LF	Acceleration or braking force
LLRM	Modified Cooper E-50 loading
LLRR	Maintenance and Construction Train (Cooper E-50)
LLV	Actual high-speed train
NSFC	Non-Standard Fastener Configuration
NUFC	Non-Uniform Fastener Configuration
PMT	Program Management Team
REJ	Rail Expansion Joint
RLD	Relative Longitudinal Displacement
RTD	Relative Transverse Displacement
RSI	Rail-Structure Interaction
RSIDAP	Rail-Structure Interaction Design and Analysis Plan
RVD	Relative Vertical Displacement
SEJ	Structural Expansion Joint
THSRC	Taiwan High Speed Rail Corporation
T_D	Temperature change of ±40˚F between rails and deck, applied to the superstructure
TSI	Track-Structure Interaction
TSIDAP	Track-Structure Interaction Design and Analysis Plan
WA	Water Loads (Stream Flow)
WS	Wind on Structure
WL_1	Wind on one 1000’ LLRM train
VTSI	Vehicle-Track-Structure Interaction
VTSIDAP	Vehicle-Track-Structure Interaction Design and Analysis Plan

1.3.2 Units

The California High-Speed Train Project (CHSTP) is based on U.S. Customary Units consistent with guidelines prepared by the California Department of Transportation (Caltrans) and defined by the National Institute of Standards and Technology (NIST). U.S. Customary Units are officially used in the U.S. and are also known in the U.S. as “English” or “Imperial” units. In order to avoid any confusion, all formal references to units of measure should be made in terms of U.S. Customary Units.

2.0 DEFINITION OF TECHNICAL TOPIC

2.1 GENERAL

This Technical Memorandum (TM) establishes California High-Speed Train Project (CHSTP) requirements for track-structure interaction (TSI) for bridges, aerial structures, grade separations, culverts, and aerial stations supporting high-speed train (HST) tracks. These structures, which are critical to ensuring elevated track performance, are referred to as “TSI-critical structures”.

TSI-critical structures are subject to the following design requirements: structural frequency recommendations, track serviceability limits, rail-structure interaction (RSI) limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction (VTSI) analysis limits.

This technical memorandum is not intended for tracks supported at grade or upon embankments. Where applicable for rail-structure interaction analyses, this technical memorandum provides guidance for modeling at-grade earthen embankments or cuts at bridge approaches and abutments.

2.1.1 CHSTP Design Considerations

The Federal Railroad Administration (FRA) issued revisions to Title 49 – Transportation, of the Code of Federal Regulations (CFR); Part 213 - Track Safety Standards, and Part 238 – Passenger Equipment Safety Standards [5]. The latest revision issued March 13, 2013 is entitled “Vehicle/Track Interaction Safety Standards; High-Speed and High Cant Deficiency Operations; Final Rule”, and creates vehicle-track safety standards applicable to high-speed and high cant deficiency train operations. Specifically, revisions to Part 213, “Subpart G- Train Operations at Track Classes 6 and Higher” form the basis for the allowable structural deformations contained within this technical memorandum.

The FRA proposed rule sets limits for track perturbations for vehicles likely to be used on high-speed or high cant deficiency rail operations. These limits are based upon results of simulation studies designed to identify track geometry irregularities associated with unsafe wheel/rail forces and accelerations. In addition, the FRA proposed rule provides a thorough review of vehicle qualification and revenue service test data with due consideration of current international practices. Different classes of track are identified based upon maximum allowable operating speed for the train; the highest of which is Class 9 track for operating speeds up to 220 mph.

Other design guidelines for high-speed facilities are under development and are defined in separate technical memoranda, including the following Technical Memoranda:

- TM 2.1.5: Track Design, which forms the basis for track design
- TM 2.3.2: Structure Design Loads, which defines loading for structures supporting high-speed trains
- TM 2.10.4: Seismic Design Criteria, which establishes seismic design for structures supporting high-speed trains

2.1.2 Design Parameters

- All structures carrying high-speed trains shall be designed to these requirements, and shall comply with the structure gauge and rail section guidelines adopted for the high-speed train system.
- The maximum initial operating speed is 220 mph. The design speed for the main tracks is 250 miles per hour; some segments of the alignment may be designed to lesser speeds.
- Bridges and aerial superstructures shall be designed to meet track serviceability limits, rail-structure interaction limits, and dynamic structural limits.
- To further ensure appropriate structural proportioning for dynamic high-speed train loads, frequency limits are provided for use in preliminary design. For final design, the

frequency limits are a recommendation only, as an extensive dynamic structural analysis is required to verify structural behavior for dynamic train passage loads.

- Per TM 2.1.5: Track Design, rail expansion joints shall be avoided since they are costly to maintain. To accomplish this, the maximum length of a structural thermal unit (i.e., the maximum distance between consecutive fixed points of thermal expansion) shall be 330 feet. In unique circumstances (e.g., long spans), rail expansion joints may be allowed with an approved design variance.
- Design and construction of high-speed train facilities shall comply with the approved and permitted environmental documents.
- At fault hazard zones (refer to TM 2.10.6: Fault Hazard Analysis and Mitigation Guidelines) bridge and aerial structures may be prohibited. Where allowed, due to the large expected fault displacements, TM 2.10.10 criteria may not be achievable. Such cases may require extensive mitigation and will be subject to the design variance approval process.

2.2 DESIGN VARIANCES TO TRACK-STRUCTURE INTERACTION DESIGN CRITERIA

Proposed design variances to the track-structure interaction criteria shall be prepared according to TM 1.1.18: Design Variance Guidelines.

Examples of performance criteria variances include:

- Exceedance of allowable deformation limits for the track and structure
- Exceedance of permissible rail stresses, under an OBE event

Examples of operational criteria variances include:

- Temporary closure for repairs following an OBE event
- Extended closures for repairs following a OBE event
- The use of rail expansion joints

2.3 PRELIMINARY DESIGN AND ANALYSIS PLAN

For preliminary design, the Designer shall develop and submit to the Authority a Preliminary Design and Analysis Plan (PDAP) for each TSI-critical structure defined as Non-standard or Complex per TM 2.10.4, or as directed by the Authority.

The PDAP shall provide an overview of the TSI-critical structure, including geometry, design constraints, and key design features. The PDAP shall define the following:

- Track Type (ballasted or non-ballasted track)
- Track Configuration (number of tracks, station components, special track features)
- Maximum Operating Speed and Design Speed
- All thermal unit lengths (LTU), defined as the point of thermal fixity to the next adjacent point of thermal fixity as described in Section 3.3.6.
- Locations and extents for all required alternative track solutions such as non-standard fastener configuration (NSFC), non-uniform fastener configuration (NUFC), or rail expansion joints (REJ) as described in Sections 3.9.6 and 3.9.7

The preliminary analysis approach for each of the applicable analysis goals in TM 2.10.4: Seismic Deisgn Criteria shall be clearly demonstrated in the PDAP. The analysis approach shall provide a summary of assumptions including, but not necessarily limited to the following:

- Mass and stiffness variations including assumed structural section properties
- The Track Fastener properties (i.e., longitudinal, vertical, and lateral stiffness)

- Model boundaries for RSI analysis
- The location and magnitude of applied live loads

In addition to issues related to TSI design, the PDAP shall be consistent with the Seismic Design and Analysis Plan (SDAP) required per TM 2.10.4.

2.4 TRACK-STRUCTURE INTERACTION DESIGN AND ANALYSIS PLAN

For final design, the Designer shall develop and submit to the Authority a Track-Structure Interaction Design and Analysis Plan (TSIDAP) for each TSI-critical structure.

The TSIDAP shall define the following:

- General Classification as Primary Type 1, Primary Type 2, or Secondary, defined per TM 2.10.4
- Technical Classification as Complex, Standard, or Non-Standard, as defined in TM 2.10.4
- Track Type (ballasted or non-ballasted track)
- Track Configuration (number of tracks, station components, special track features)
- The Track Fastener properties assumed for analysis (i.e., longitudinal, vertical, and lateral stiffness)
- The approach used to determine the model boundaries for RSI analysis per Section 3.9.8
- The approach for developing vertical and lateral track stiffness properties at adjacent earthern embankments or cuts per Section 3.9.9
- Maximum Operating Speed and Design Speed
- The span arrangement layout in compliance with requirements in TM 2.3.2
- Thermal unit lengths (L_{TU}), defined as the point of thermal fixity to the next adjacent point of thermal fixity as described in Section 3.3.6.
- Locations and extents for all required alternative track solutions such as non-standard fastener configuration (NSFC), non-uniform fastener configuration (NUFC), or rail expansion joints (REJ) as described in Sections 3.9.6 and 3.9.7

The TSIDAP shall be consistent with the Seismic Design and Analysis Plan (SDAP) required per TM 2.10.4.

The TSIDAP shall contain detailed commentary on track-structure interaction analysis for applicable analysis goals, indicating the analysis software to be used, the modeling assumptions, and techniques to be employed.

The TSIDAP shall include an outline of analysis modeling requirements including mass and stiffness variations, presence of continuous welded rail, and live load configurations. A detailed approach for development of model boundaries at foundations, embankments, and continuous welded rail model boundaries shall also be provided.

For dynamic structural analysis per Section 3.7, the TSIDAP shall summarize the approach for determination of resonance speeds, including the design iteration approach for any structures not consisting entirely of simple spans. Techniques for determining dynamic impact factors and vertical deck accelerations shall be included.

The TSIDAP shall discuss the approach for determining the rail-structure interaction forces caused by creep, shrinkage, prestressing, and temperature effects per TM 2.3.2. The approach for implementation of results into Strength and Service Load combinations in TM 2.3.2 shall be provided.

To meet RSI criteria per Section 3.6, the TSIDAP may include proposals for alternative track solutions (e.g., NSFC, NUFC, REJs, etc.) through the design variance approval process. The design variance shall be supplemented with a special RSI analysis per Section 3.9.7.

As determined by the Authority, advanced supplemental plans may be required as part of the TSIDAP. These advanced supplemental plans are to be required as part of conditional approval for design variance requests or for those critical Complex structures which depart from current service-proven design concepts. Advanced supplemental plans include, but are not limited to:

- Vehicle-Track-Structure Interaction Design and Analysis Plan (VTSIDAP) per Section 3.8.1
- Rail-Structure Interaction Design and Analysis Plan (RSIDAP) per Section 3.9.7

Track-structure interaction related design variances shall be submitted per the project design variances guidelines. The TSIDAP shall justify all track-structure interaction design variances related to track performance, rail-structure interaction, or dynamic structural response.

2.5 SEISMIC DESIGN AND ANALYSIS PLAN

For final design, as a requirement of TM 2.10.4: Seismic Design Criteria, the designer shall develop and submit a Seismic Design and Analysis Plan (SDAP) to the Authority.

As part of this TM, seismic analysis and design for the Operating Basis Earthquake (OBE) is required. The SDAP shall discuss in detail the proposed analysis for the OBE, indicating the analysis software to be used as well as the modeling assumptions made and the various modeling techniques to be employed.

Analysis software packages shall conform to CHSTP software verification and quality assurance guidelines. Analysis software shall be capable of non-linear analysis capabilities as required for TM 2.10.10: Track Structure Interaction and TM 2.10.4: Seismic Design Criteria.

A description of SDAP requirements is provided in TM 2.10.4.

2.6 DESIGN REFERENCES AND CODES

As stated in Section 2.1.1, revisions to Title 49 – Transportation, of the Code of Federal Regulations (CFR); Part 213 - Track Safety Standards, and Part 238 – Passenger Equipment Safety Standards [5] form the basis for the allowable structural deformations contained within this technical memorandum.

This technical memorandum also uses guidance drawn from the following design references and codes:

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009 [1]
2. European Standard EN 1991-2:2003 Traffic Loads on Bridges [2]
3. European Standard EN 1990:2002 + A1:2005 Basis of Structural Design [3]
4. Taiwan High Speed Rail (THSR) Corporation Design Specifications [4]
5. People's Republic of China, Code for Design of High-Speed Railway (2009) [23]
6. Japanese Standard 2007.03 - Design Standards for Railway Structures and Commentary (Displacement Limits) [24]

Criteria for design elements not specific to high speed rail operations will be governed by existing applicable standards, laws and codes. Applicable local building, planning and zoning codes and laws are to be reviewed for the stations, particularly those located within multiple municipal jurisdictions, state rights-of-way, and/or unincorporated jurisdictions.

In the case of differing values, the standard followed shall be that which results in the satisfaction of all applicable requirements. In the case of conflicts, documentation for the conflicting standard

is to be prepared and approval is to be secured as required by the affected agency for which an exception is required, whether it be an exception to the CHSTP standards or other agency standards.

Design shall meet all applicable portions of the general laws and regulations of the State of California and of respective local authorities.

3.0 ASSESSMENT / ANALYSIS

3.1 GENERAL

Bridges and aerial structures that support high-speed trains are subject to the following frequency limits, track serviceability limits, rail-structure interaction limits, and passenger comfort limits.

These requirements are concerned with limiting bridge and aerial structure deformations and accelerations, since the structure response can be dynamically magnified under high-speed moving trains. Excessive deformations and accelerations can lead to numerous issues, including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

Preliminary and final design level analysis requirements are given in Section 3.2.

Table 3-1 summarizes the analysis requirements, including model type, train model/speed, result, and relevant subsections.

Table 3-1: Track-Structure Interaction Analysis Requirements – Preliminary Design

Analysis Goal	Model Type	Train model	Train speed	Result	Subsection(s)
Frequency Analysis	Dynamic	--	--	Frequency Evaluation	3.4.2, 3.4.3, 3.4.4
Track Serviceability Analysis	Static, For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation Limits	3.5.2 to 3.5.11
Rail-Structure Interaction Analysis	Static (linear or non-linear), For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation And Rail Stress Limits	3.6.2 to 3.6.7
Dynamic Structural Analysis	Dynamic	Single Track of High-Speed Train Passage	90 mph to 1.2 Line Speed (or 250 mph whichever is less)	Dynamic Impact Factor, Vertical Deck Acceleration	3.7.2 to 3.7.5
Dynamic Vehicle-Track-Structure Interaction Analysis	Dynamic (Structure & Trainset)	Single Track of High-Speed Passage (with Vehicle Suspension)	90 mph to 1.2 Line Speed (or 250 mph whichever is less)	Dynamic Track Safety and Passenger Comfort Limits	3.8.2 to 3.8.4

3.2 ANALYSIS REQUIREMENTS

3.2.1 General

As part of LRFD force-based design, static analysis is required for load combinations including Cooper E-50 maintenance and construction trains (LLRR), actual high-speed train loads (LLV), and vertical impact effects (I and I_{LLV}). Refer to TM 2.3.2: Structural Design Loads.

Additional analyses are required beyond what is provided for in this technical memorandum in order to determine basic structural proportioning, ensure track safety, and provide passenger comfort for high-speed train operation on TSI-critical structures.

3.2.2 Preliminary Design Requirements

Frequency analysis, track serviceability analysis, and rail-structure interaction (RSI) analysis shall apply for preliminary design of TSI-critical structures.

For preliminary design, where a TSI-critical structure falls above the recommended lower bound frequency thresholds (Section 3.4), the following is applicable:

- For Standard, Non-Standard and Complex structures per TM 2.10.4, dynamic structural analysis of high-speed train passage shall not be required.
- Dynamic vehicle-track-structure interaction (VTSI) analysis per Section 3.8 shall not be required.

For preliminary design, where a TSI-critical structure falls below the recommended lower bound frequency thresholds (Section 3.4), the following is applicable:

- For Standard, Non-Standard, and Complex structures per TM 2.10.4, a limited dynamic structural analysis of high-speed train passage per Section 3.7.2 at selected speeds (as provided in Section 3.7.3) shall be required.
- Dynamic VTSI analysis described in Section 3.8 shall not be required.

3.2.3 Final Design Requirements

Frequency analysis, track serviceability analysis, rail-structure interaction (RSI) analysis, and a full dynamic structural analysis of high-speed train passage per Section 3.7.2, shall apply for final design for all structures (Standard, Non-Standard, and Complex structures as described in TM 2.10.4).

For final design, the recommended lower bound frequency thresholds are provided for reference only. The frequency recommendations serve as a means for the initial assessment of dynamic performance. The vertical, lateral, and torsional frequencies in Section 3.4 must be recorded for reference on the final design plans.

For final design, the full dynamic structural analysis of high-speed train passage per Section 3.7 provides the primary assessment of dynamic performance.

Dynamic VTSI analysis per Section 3.8 may be required, as determined by the Authority, for those TSI-critical structures not in compliance with the deformation and acceleration requirements in Sections 3.5 through 3.7. Dynamic VTSI also may be required as determined by the Authority for those critical Complex structures per TM 2.10.4 departing from service-proven concepts.

For final design of non-ballasted TSI-critical structures, RSI analysis shall be supplemented with an additional requirement to analyze uplift at direct-fixation fasteners per Section 3.6.6.

3.3 DESIGN PARAMETERS

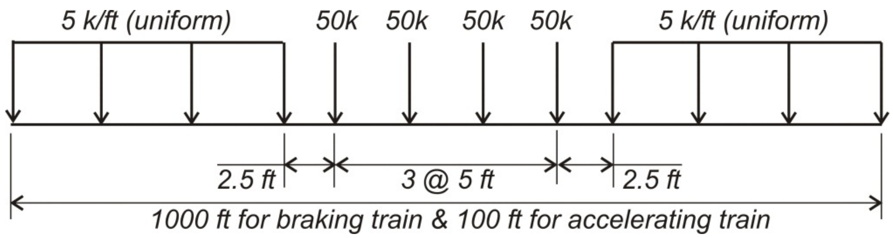
3.3.1 General

The following defines loading to be used for track serviceability and rail-structure interaction analysis.

3.3.2 Modified Cooper E-50 Loading (LLRM)

Modified Cooper E-50 loading (LLRM) per Figure 3-1 shall be used for track serviceability analysis per Section 3.5, and rail-structure interaction analysis per Section 3.6. LLRM loading is on a per track (i.e., two rail) basis.

Figure 3-1: LLRM Loading



3.3.3 Vertical Impact Effect (I)

The vertical impact effect (I) used with Modified Cooper E-50 loading (LLRM) shall be vertical impact effect from LLRR per TM 2.3.2: Structure Design Loads.

Dynamic vertical impact effects (I_{LLV}) caused by high-speed trainsets (LLV) shall be found per Section 3.7.4.

3.3.4 Centrifugal Force (CF)

The centrifugal force (CF) used with Modified Cooper E-50 loading (LLRM) shall be determined per TM 2.3.2: Structure Design Loads. The maximum CF calculated for LLRR and LLV shall be used, whichever governs.

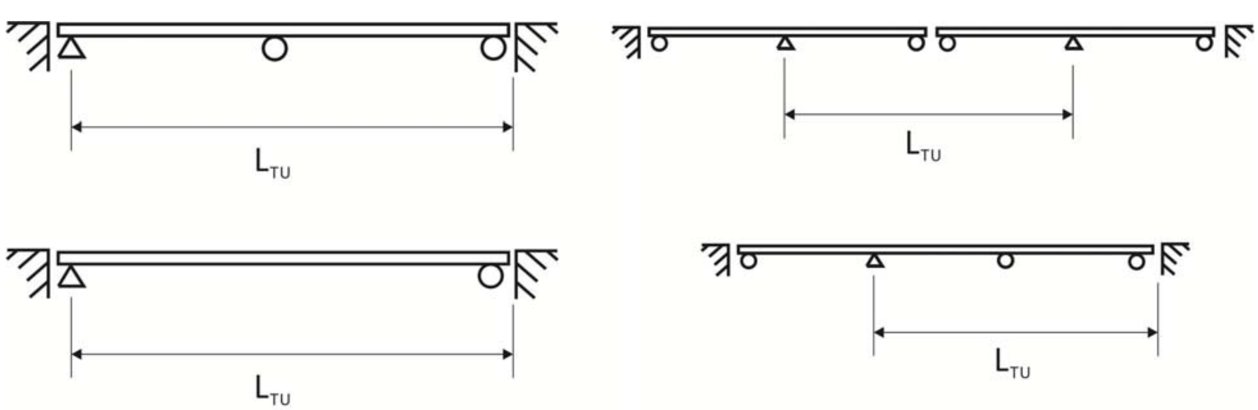
3.3.5 Accelerating and Braking Force (LF)

The longitudinal accelerating and bracking forces (LF) used with Modified Cooper E-50 loading (LLRM) shall be determined using the approach for LLV loading per TM 2.3.2: Structure Design Loads.

3.3.6 Structural Thermal Unit

Per TM 2.1.5: Track Design, rail expansion joints shall be avoided since these are costly to maintain. To accomplish this, the maximum length of a structural thermal unit (L_{TU}), defined as the distance between consecutive fixed points of thermal expansion, shall be 330 feet. Refer to Figure 3-2.

Figure 3-2: Structural Thermal Unit



In unique circumstances (e.g., long spans), rail expansion joints may be allowed with an approved design variance and the additional requirement of special rail-structure interaction analysis per Section 3.9.7.

3.4 FREQUENCY ANALYSIS

3.4.1 General

Frequency limits are placed on the fundamental mode shapes of TSI-critical structures, in order to ensure well-proportioned structures and minimize resonancy effects.

Upper and lower bound mass and stiffness assumptions shall be evaluated per the modeling requirements as given in Section 3.9.

3.4.2 Recommended Threshold of Vertical Frequency of Span

The recommended vertical lower bound frequency threshold is known to favorably resist high-speed train resonance actions. It is recommended that structures be proportioned to fall above this lower bound threshold.

Where a structure falls below the recommended vertical frequency threshold, then additional analysis shall be required per Section 3.2.

Vertical frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For vertical frequency analysis, two conditions must be investigated:

- Condition #1: a lower bound estimate of stiffness and upper bound estimate of mass
- Condition #2: an upper bound estimate of stiffness and lower bound estimate of mass

Condition #1 will govern the lower bound threshold. Condition #2 is required for future structural assessment.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 3.9.

The recommended threshold for the first natural frequency of vertical deflection, η_{vert} [Hz], primarily due to bending of the span is the following:

$$\eta_{\text{vert}} \geq \eta_{\text{lower}}$$

Where:

$\eta_{lower} = 313.09L^{-0.917}$ for $L \leq 330$ feet

where L = effective length of span (feet)

For simple spans, L shall be the span length.

For continuous spans, L shall be the following:

$$L = k\left(L_{\text{average}}\right)$$

Where:

$$L_{\text{average}} = \frac{(L_1 + L_2 + ... + L_n)}{n}$$ = the average span length

$n =$ the number of spans

$$k = \left(1 + \frac{n}{10}\right) \leq 1.5$$

For portal frames and closed frame bridges, L shall be:

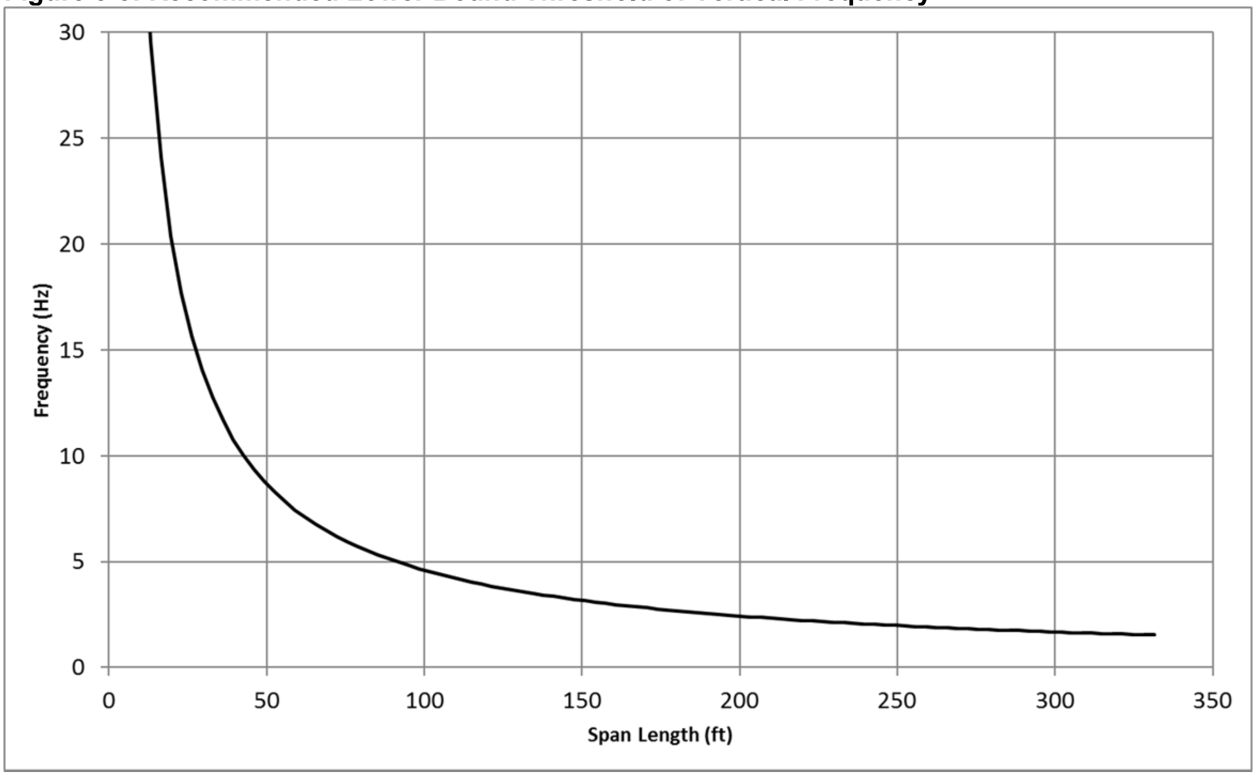
- Single span: consider as three (3) continuous spans, with the first and third span being the vertical length of the columns, and the second span the girder length.
- Multiple spans: consider as multiple spans, with the first and last span as the vertical length of the end columns, and the interior spans the girder lengths.

For spans with end diaphragms at abutments (fixed supports at abutments), the following L shall apply:

- Single span, fixed at one abutment: consider as two (2) continuous spans, with the first span equal to 0.05 times the girder length, and the second span the girder length.
- Single span, fixed at both abutments: consider as three (3) continuous spans, with the first and the third span equal to 0.05 times the girder length, and the second span the girder length.
- Multiple spans, fixed at one abutment: consider as multiple spans, with the first span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.
- Multiple spans, fixed at both abutments: consider as multiple spans, with the first and last span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.

For single arch, archrib, or stiffened girders of bowstrings, L shall be the half span. Refer to Figure 3-3 for the recommended lower bound threshold of vertical frequency.

Figure 3-3: Recommended Lower Bound Threshold of Vertical Frequency



3.4.3 Recommended Lower Bound Torsional Frequency of Span

Recommendations for lower bound torsional frequency are to proportion structures to favorably resist high-speed train actions.

All torsional frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For torsional frequency analysis, two conditions must be investigated, consistent with vertical frequency analysis:

- Condition #1 – a lower bound estimate of stiffness and upper bound estimate of mass
- Condition #2 – an upper bound estimate of stiffness and lower bound estimate of mass

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 3.9.

For Conditions #1 and #2, the first torsional frequency, $\eta_{torsion}$, of the span shall be greater than 1.2 times the corresponding first natural frequency of vertical deflection, η_{vert} [2].

3.4.4 Recommended Lower Bound Transverse Frequency of Span

Recommendations for the lower bound transverse frequency favorably resists high-speed train actions.

For transverse frequency analysis, two conditions shall be investigated:

- Condition #1 – consideration of flexibility of superstructure only, excluding the flexibility of bearings, columns, and foundations, assuming supports at ends of the span are rigid [8].
- Condition #2 – consideration of flexibility of superstructure and substructure, including flexibility of bearings, columns, shear keys, and foundations.

For transverse frequency analysis, a lower bound estimate of stiffness and upper bound estimate of mass shall be used, refer to Section 3.9.

For Condition #1, the first natural frequency of transverse deflection, η_{trans}, of the span shall not be less than 1.2 Hz [3].

For Condition #2, no frequency recommendation is provided, but shall be recorded for future structural assessment.

3.5 TRACK SERVICEABILITY ANALYSIS

3.5.1 General

Track serviceability analysis, using modified Cooper E-50 loading, provides limits to allowable structural deformations. These track serviceability limits are developed for structures supporting continuous welded rail without rail expansion joints.

Deformation limits are developed for limit states based on maintenance, passenger comfort, and track safety requirements. For information into the development of these limits, see the following project specific white papers:

- "Track Serviceability Structural Deformation Limits – Profile" [18]
- "Track Serviceability Structural Deformation Limits – Alignment" [19]
- "Track Serviceability Structural Deformation Limits – Deck Twist" [20].

For track serviceability analysis, the flexibility of superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered.

In order to avoid underestimating deformations, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Details of modeling requirements are given in Section 3.9.

3.5.2 Track Serviceability Load Cases

Track serviceability loads cases shall include [4] :

- Group 1a: $(LLRM + I)_1 + CF_1 + WA$
- Group 1b: $(LLRM + I)_2 + CF_2 + WA$
- Group 1c: $(LLRM + I)_m + CF_m + WA$
- Group 2: $(LLRM + I)_1 + CF_1 + WA + WS + WL_1$
- Group 3: $(LLRM + I)_1 + CF_1 + OBE$

where:

$(LLRM + I)_1$ = one track of (LLRM + I) plus impact

$(LLRM + I)_2$ = two tracks of (LLRM + I) plus impact

$(LLRM + I)_m$ = multiple tracks per Section 3.5.5 of (LLRM + I) plus impact

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (one track) per TM 2.3.2: Structure Design Loads

CF_2 = centrifugal force (two tracks) per TM 2.3.2: Structure Design Loads

CF_m = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

WA = water loads (stream flow) per TM 2.3.2: Structure Design Loads

WS & WL_1 = wind on structure and wind on one 1000' LLRM train per TM 2.3.2: Structure Design Loads

OBE = Operating Basis Earthquake per TM 2.10.4: Seismic Design Criteria

Note that Group 1c is used for Section 3.5.5 only.

Static analysis and linear superposition of results shall be allowed for Groups 1b, 1c, and 2.

For determining OBE demands in Group 3, equivalent static analysis, dynamic response spectum, or time history (linear or non-linear) analysis may be used, in accordance with the approved PDAP per Section 2.3 and SDAP per Section 2.4. Refer to TM 2.10.4: Seismic Design Criteria for additional OBE modeling requirements.

For track serviceability analysis, non-linear track-structure interaction modeling (refer to Section 3.9.6) is not required, but may be used. For Group 3, superposition of static (i.e., $(LLRM + I)_1$ + CF_1) and either static or dynamic OBE shall be allowed.

3.5.3 Vertical Deflection Limits: Group 1a

Vertical deflection limits for Group 1a are to address maintenance, passenger comfort, and track safety issues.

For Group 1a, the maximum static vertical deck deflection (max Δ_{1a}), in the most unfavorable position, shall not exceed the limits given in Table 3-2 [18].

Table 3-2: Vertical Deflection Limits: Group 1a

Limit	Span Length				
	L ≤ 125ft	L=175ft	L=225ft	L=275ft	L≥330ft
max Δ_{1a}	L/3500	L/3180	L/2870	L/2550	L/2200

Note: Limits apply for both non-ballasted and ballasted track

For span lengths not explicitly referenced in Table 3-2, use linear interpolation.

3.5.4 Vertical Deflection Limits: Group 1b

Vertical deflection limits for Group 1b are to address maintenance, passenger comfort, and track safety issues.

For Group 1b, the maximum static vertical deck deflection (max Δ_{1b}), in the most unfavorable position, shall not exceed the limits given in Table 3-3 [18].

Table 3-3: Vertical Deflection Limits: Group 1b

Limit	Span Length				
	L ≤ 125ft	L=175ft	L=225ft	L=275ft	L≥330ft
max Δ_{1b}	L/2400	L/2090	L/1770	L/1450	L/1100

Note: Limits apply for both non-ballasted and ballasted track

For span lengths not explicitly referenced in Table 3-3, use linear interpolation.

3.5.5 Vertical Deflection Limits: Group 1c

Vertical deflection limits for Group 1c are to provide practical guidance for structures containing three or more tracks operating at speeds less than 90 mph. This guidance is consistent with established European codes.

$(LLRM + I)_m$ and CF_m loading shall be applied in a manner consistent with the case of multiple tracks on structures as described below:

- For 2 tracks, full live load on 2 tracks.
- For 3 tracks, full live load on 2 tracks and one-half on the other track.
- For 4 tracks, full live load on 2 tracks, one-half on one track, and one-quarter on the remaining one.
- More than 4 tracks shall be considered on an individual basis.

The tracks selected for loading shall be those tracks which will produce the most critical design condition on the member under consideration.

For Group 1c, where the structures support three or more tracks, the maximum static vertical deck deflection (max Δ_{1c}), in the most unfavorable position, shall not exceed L/600 for all span lengths [3]. This limit applies for both non-ballasted and ballasted track.

In the event that structures support 3 or more tracks, and 3 or more trains can be anticipated to be on the same structure at speeds greater than 90 mph, limits defined for Group 1b shall apply. For these structures, representative live load conditions must be developed on a case-by-case basis.

3.5.6 Transverse Deflection Limits

Transverse deflection limits are to address maintenance, passenger comfort, and track safety issues.

The transverse deflection within the span (Δ_{trans}), shown in Figure 3-4, shall not exceed the limits given in Table 3-4 [19].

Figure 3-4: Transverse Span Deformation Limits

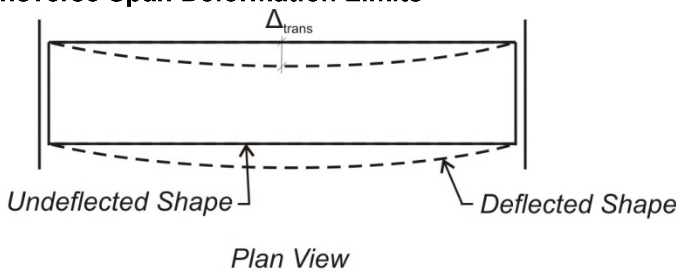


Table 3-4: Transverse Deflection Limits

Group	Δ_{trans} (feet)
1a	$L^2/(864{,}800)$
1b	$L^2/(447{,}200)$
2	$L^2/(276{,}800)$
3	$L^2/(276{,}800)$

Note: Limits apply for both non-ballasted and ballasted track

3.5.7 Rotation about Transverse Axis Limits

Rotation about transverse axis limits are to control excessive rail axial and bending stress, provide traffic safety (i.e., guard against wheel unloading due to abrupt angular changes in track geometry), and provide passenger comfort.

Due to rotation about the transverse axis, imposed axial rail displacement is a linear function of the distance between the rail centroid and top of the bridge bearings. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about transverse axis at deck ends (θ_t), shown in Figure 3-5, shall be defined by the following equations:

$\theta_t = \theta$, for abutment condition

$\theta_t = \theta_1 + \theta_2$, between consecutive decks

Also, the maximum relative axial displacement at the rail centroid (δ_t) due to rotation about transverse axis, shown in Figure 3-5, shall also be defined by the following equations:

$\delta_t = \theta h$, for abutment condition

$\delta_t = \delta_1 + \delta_2 = \theta_1 h_1 + \theta_2 h_2$, between consecutive decks.

where:

θ_t (radians): total rotation about transverse axis; see Table 3-5 [18]

δ_t (in): total relative displacement at the rail centroid; see Table 3-5 [18]

θ (radians): rotation of the bridge bearing at abutment

θ_1 (radians): rotation of the first bridge bearing

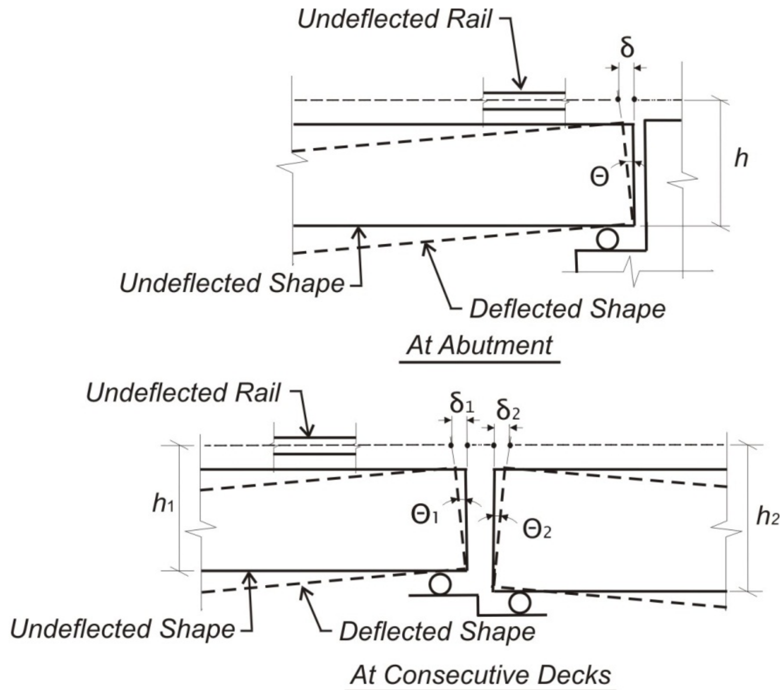
θ_2 (radians): rotation of the second bridge bearing

h (in): the distance between the rail centroid and the bridge bearing at abutment

h_1 (in): the distance between the rail centroid and the top of the first bridge bearing

h_2 (in): the distance between the rail centroid and the top of the second bridge bearing

Figure 3-5: Rotation about Transverse Axis at Deck Ends



The total rotation about transverse axis (θ_t) and the total relative displacement at the rail centroid (δ_t) shall not exceed the limits given in Table 3-5. [18].

Table 3-5: Rotation about Transverse Axis and Relative Displacement at the Level of the Rail Limits

Group	θ_t (radians)	δ_t (inches)	
		Non-ballasted Track	Ballasted Track
1a	0.0012	0.33	0.33
1b	0.0017	0.33	0.33
2	0.0026	0.67	0.67
3	0.0026	0.67	0.67

3.5.8 Rotation about Vertical Axis Limits

Rotation about vertical axis limits are to control rail axial and bending stress, provide track safety, and provide passenger comfort by limiting changes in horizontal track geometry at bridge deck ends.

Due to rotation about the vertical axis, imposed longitudinal rail displacement is a linear function of the distance between the centerline of span and the outermost rail. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about vertical axis at deck ends (θ_v), shown in Figure 3-6, shall be defined by the following equations:

$\theta_v = \theta$, for abutment condition

$\theta_v = \theta_A + \theta_B$, between consecutive decks

The maximum axial displacement at the outermost rail centroid (δ_v) due to rotation about vertical axis, shown in Figure 3-7, shall be defined by the following equations:

$\delta_v = \theta w$, for abutment condition

$\delta_v = \delta_A + \delta_B = \theta_A w_A + \theta_B w_B$, between consecutive decks.

where:

θ_v (radians): total rotation about vertical axis; see Table 3-6

θ (radians): rotation of the bridge at abutment

θ_A (radians): rotation of the first span

θ_B (radians): rotation of the second span

w (in): the distance between the centerline span and outermost rail centroid at abutment

w_A (in): the distance between the centerline span and outermost rail centroid of first span

w_B (in): the distance between the centerline span and outermost rail centroid of second span

δ_v (in): total relative displacement at the outermost rail centroid; see Table 3-6

δ (in): relative displacement at the outermost rail centroid, at abutment

δ_A (in): relative displacement at the outermost rail centroid, first span

δ_B (in): relative displacement at the outermost rail centroid, second span

Figure 3-6: Rotation about Vertical Axis at Deck Ends – Global View

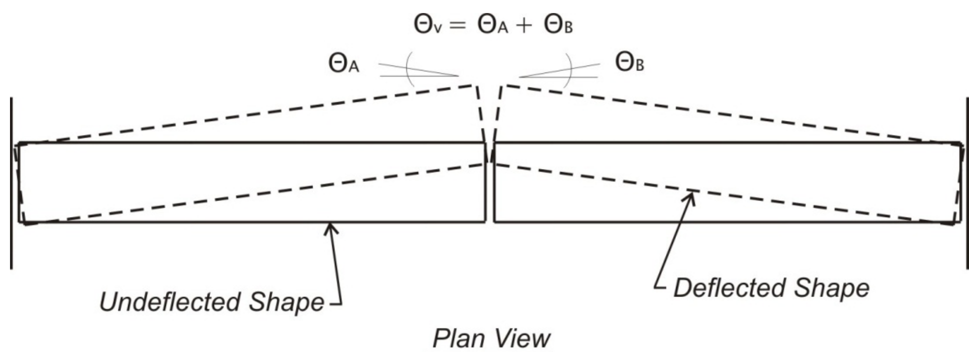
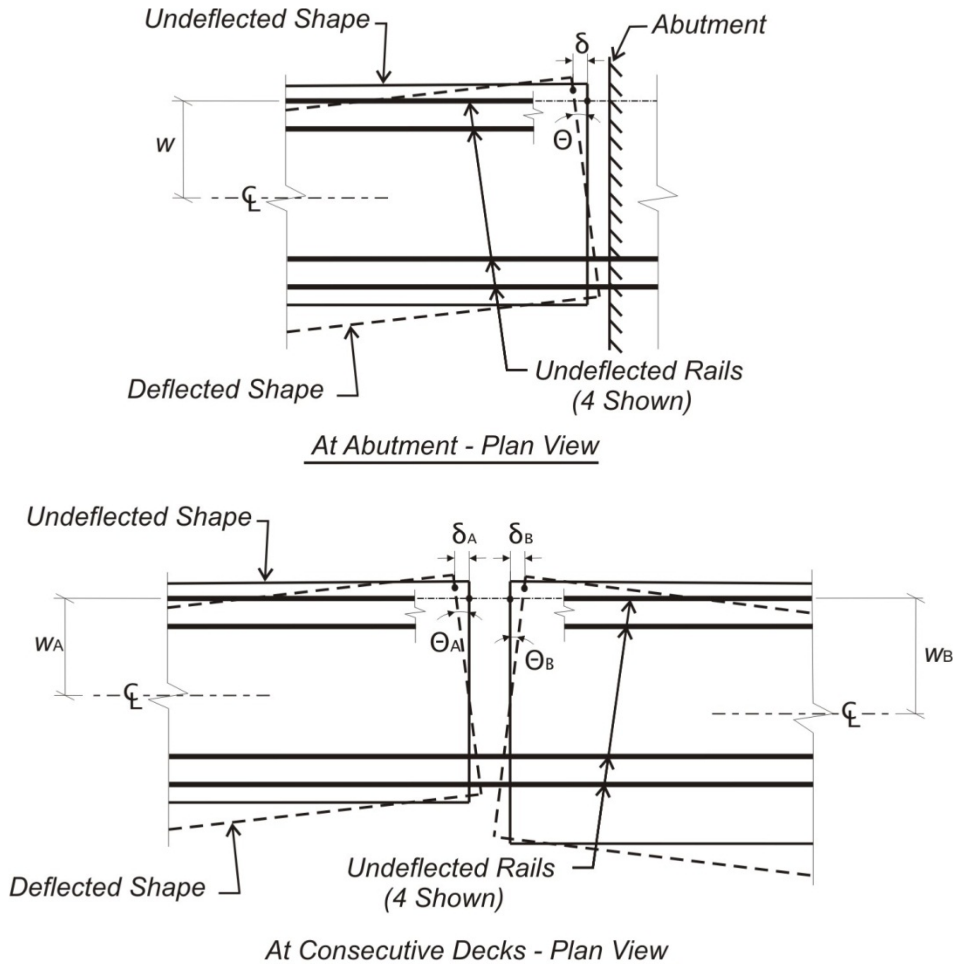


Figure 3-7: Rotation about Vertical Axis at Deck Ends – Local View



The total rotation about vertical axis (θ_v) and the total relative displacement at the outermost rail centroid (δ_v) shall not exceed the limits given in Table 3-6, as developed in [19].

Table 3-6: Rotation about Vertical Axis and Relative Displacement at Outermost Rail Limits

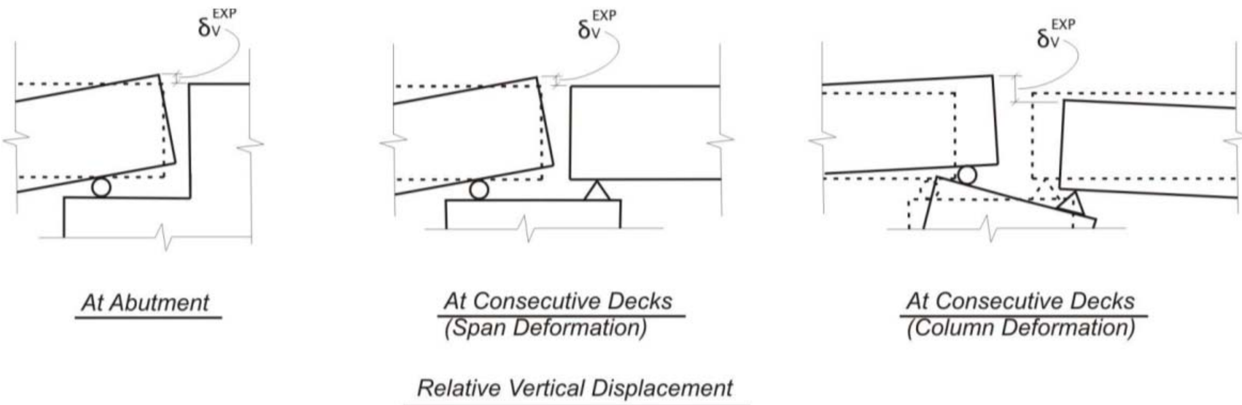
Group	θ_v (radians)	δ_v (inches)	
		Non-ballasted Track	Ballasted Track
1a	0.0007	0.33	0.33
1b	0.0010	0.33	0.33
2	0.0021	0.67	0.67
3	0.0021	0.67	0.67

3.5.9 Relative Vertical Displacement at Expansion Joints – Track Serviceability

Relative vertical displacements (RVD) at structural expansion joints, δ_V^{EXP}, are limited in order to ensure track safety subject to deck end rotation and vertical bearing deformation. As shown in Figure 3-8, structural expansion joints between adjacent deck ends, and between deck ends and abutments, shall be considered.

The flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

Figure 3-8: Relative Vertical Displacement at Expansion Joints – Track Serviceability



The RVD at expansion joints (δ_V^{EXP}), shall not exceed the limits given in Table 3-7.

Table 3-7: Relative Vertical Displacement at Expansion Joints Limits – Track Serviceability

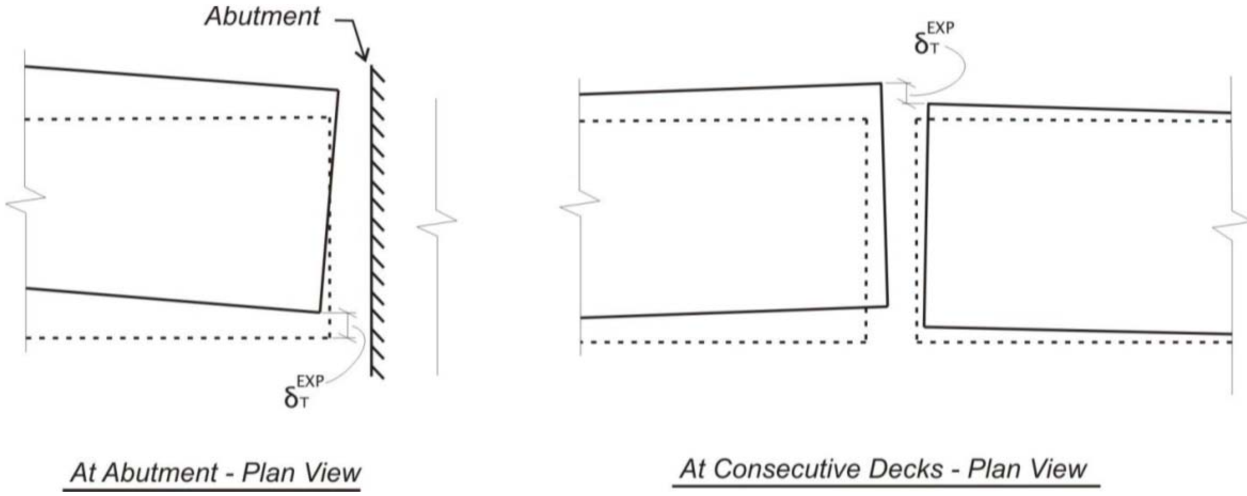
Group	δ_V^{EXP} (inches)
1a	0.25
1b	0.25
2	-
3	-

Note: Limits apply for both non-ballasted and ballasted track

3.5.10 Relative Transverse Displacement at Expansion Joints – Track Serviceability

Relative transverse displacements (RTD) at structural expansion joints, δ_T^{EXP}, are limited in order to ensure track safety subject to shear key and lateral bearing deformation. As shown in Figure 3-9, structural expansion joints between adjacent deck ends, and between deck ends and abutments, shall be considered.

Figure 3-9: Relative Transverse Displacement at Expansion Joints – Track Serviceability



The RTD at expansion joints (δ_T^{EXP}) shall not exceed the limits given in Table 3-8.

Table 3-8: Relative Transverse Displacement at Expansion Joints Limits – Track Serviceability

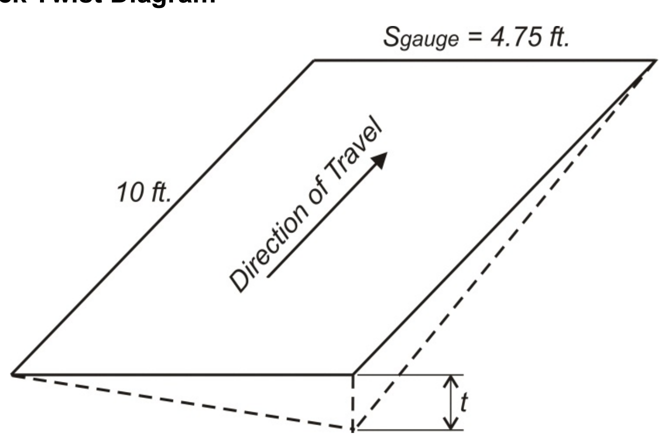
Group	δ_T^{EXP} (inches)
1a	0.08
1b	0.08
2	-
3	-

Note: Limits apply for both non-ballasted and ballasted track

3.5.11 Deck Twist Limits

The deck twist, t, is defined as the relative vertical deck displacment of a given bogie contact point from a plane defined by the remaining three bogie contact points on a track gauge of 4.75 feet over a bogie length of 10 feet, refer to Figure 3-10. Deck twist limits ensure that the four wheel contact points of a bogie are not too far from a plane.

Figure 3-10: Deck Twist Diagram



Maximum deck twist (t_{max}) below tracks shall not exceed the limits given in Table 3-9 [20].

Table 3-9: Deck Twist Limits

Group	t_{max} (in/10 feet)
1a	0.06
1b	0.06
2	0.17
3	0.17

Note: Limits apply for both non-ballasted and ballasted track

3.6 RAIL-STRUCTURE INTERACTION ANALYSIS

3.6.1 General

Rail-structure interaction (RSI) analysis, using modified Cooper E-50 loading (LLRM), shall be used to limit relative longitudinal, vertical, and transverse displacements at structural expansion joints, and limit axial rail stress in order to minimize the probability of rail fracture. Deformation and rail stress limits were developed considering the accumulation of displacement demands and rail bending stresses under the controlling load combinations.

Details of RSI modeling requirements are given in Section 3.9.6.

For RSI analysis, the flexibility of superstructure, bearings, shear keys, columns, and foundations shall be considered.

For all RSI analysis, in order to avoid underestimating deformations and rail stress, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Limits on expansion joint displacement, fastener performance, and rail stress are provided in Sections 3.6.3 through 3.6.7. These limits only apply if all assumptions and modeling requirements given in Section 3.9.6 are valid. For structures requiring alternative assumptions or modeling techniques, an approved design variance and a special RSI analysis per Section 3.9.7 shall be required.

Deformation limits and rail stress limits were developed considering the accumulation of displacement demands and rail bending stresses under the controlling load combinations. For more information into the development of these limits, see the project specific white paper entitled, “Rail Stress Evaluation and Fastener Restraint” [21].

See Section 3.2 to determine when rail-structure interaction analysis is required for preliminary and final design.

3.6.2 Rail-Structure Interaction Load Cases

Rail-structure interaction (RSI) load cases include the following [4]:

- Group 4: $(LLRM + I)_2 + LF_2 \pm T_D$
- Group 5: $(LLRM + I)_1 + LF_1 \pm 0.5T_D + OBE$

where:

$(LLRM + I)_1$ = single track of Modified Cooper E-50 (LLRM) plus vertical impact effect

$(LLRM + I)_2$ = two tracks of Modified Cooper E-50 (LLRM) plus vertical impact effect

I = vertical impact effect from LLRR per TM 2.3.2: Structure Design Loads

LF_1 = braking forces (apply braking to one track) for LLV loading per TM 2.3.2: Structure Design Loads

LF_2 = braking and acceleration forces (apply braking to one track, acceleration to the other track) for LLV loading per TM 2.3.2: Structure Design Loads

T_D = temperature differential of ±40˚F between rails and deck, applied to the superstructure

OBE = Operating Basis Earthquake per TM 2.10.4: Seismic Design Criteria

Groups 4 and 5 are to provide relative longitudinal, vertical, and transverse displacement limits at expansion joints, and design for uplift at direct fixation rail. Groups 4 and 5 are also used to limit rail stress, accounting for thermal effects (i.e. ± T_D).

Modeling of nonlinear RSI effects, as given in Section 3.9.6, shall be required to give realistic demands. Experience has shown that linear modeling of RSI is overly conservative.

For Group 5, non-linear time-history OBE analysis (i.e., non-linear RSI) shall be used for design. $(LLRM + I)_1 + LF_1$ may be idealized as a set of stationary load vectors placed upon the structure in the most unfavorable position. Refer to TM 2.10.4: Seismic Design Criteria for additional OBE modeling requirements.

3.6.3 Relative Longitudinal Displacement at Expansion Joints

Relative longitudinal displacements (RLD) at structural expansion joints, δ_L^{EXP}, are limited in order to prevent excessive rail axial stress. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

RLD at structural expansion joints, δ_L^{EXP}, has components due to both structural translation and structural rotation. For structural rotation, RLD is a function of distance from center of structure rotation to the rail centroid. Therefore, δ_L^{EXP} shall be monitored relative to the original rail centroid location, and consist of structural movement alone.

δ_L^{EXP}, determined at the rail centroid, consists of separate components:

- δ_{LF} = component due to acceleration and braking only, refer to Figure 3-11
- δ_{LLRM+I} = component due to vertical train plus impact loads only, refer to Figure 3-12
- δ_{OBE} = component due to OBE only (refer to Figure 3-13), comprised of:
    - $\delta_{OBE(L)}$ = longitudinal displacement subcomponent due to OBE
    - $\delta_{OBE(V)}$ = rotation about vertical axis subcomponent due to OBE
    - $\delta_{OBE(T)}$ = rotation about transverse axis subcomponent due to OBE
    - $\delta_{OBE} = \delta_{OBE(L)} + \delta_{OBE(V)} + \delta_{OBE(T)}$
- δ_{TD} = component due to temperature differential (T_D) between superstructure and rail

Figure 3-11: δ_{LF} definition

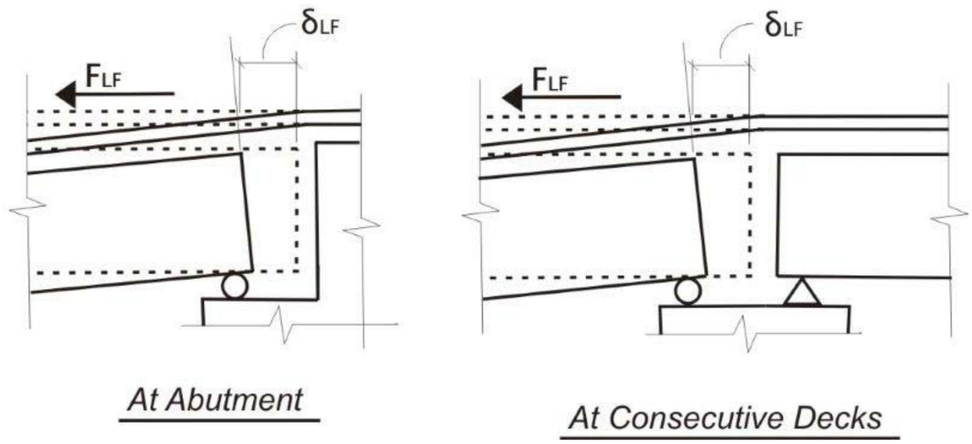


Figure 3-12: δ_{LLRM+I} definition

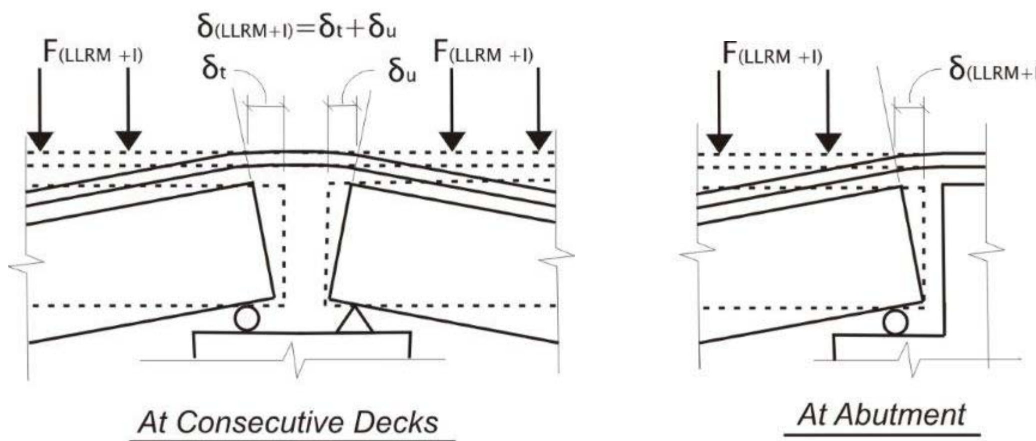
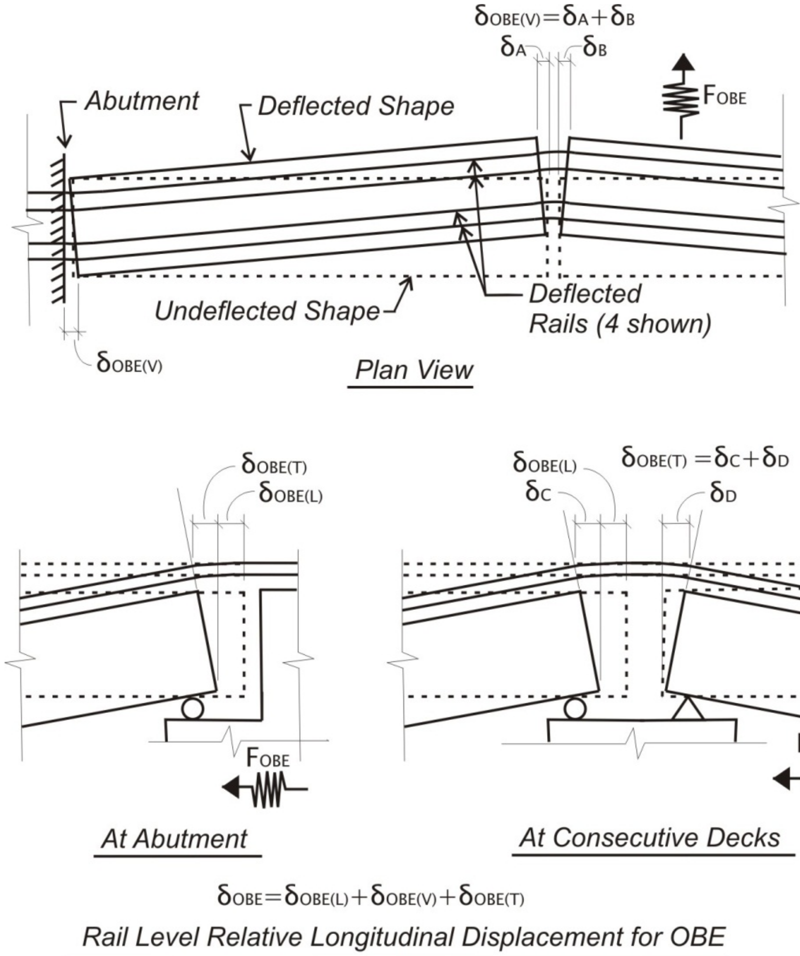


Figure 3-13: δ_{OBE} definition



The RLD at expansion joints measured relative to the original rail centroid locations (δ_L^{EXP}) shall not exceed the limits given in Table 3-10.

Note that in order to prevent having separate load cases for relative displacement and rail stress design, the expected temperature differential demands are added to the displacement limits. The temperature differential demands are dependent on the structural thermal unit (L_{TU}), which is defined as the point from fixed point of thermal expansion to the next adjacent fixed point of thermal expansion. The maximum L_{TU} shall not exceed 330 feet without an approved design variance and special RSI analysis per Section 3.9.7.

Table 3-10: Relative Longitudinal Displacement at Expansion Joints Limits

Group	δ_L^{EXP} (inch)	
	Non-ballasted	Ballasted
4	$0.70 + \delta_{TD,Expected}$	$0.50 + \delta_{TD,Expected}$
5	$2.33 + 0.5\delta_{TD,Expected}$	$2.25 + 0.5\delta_{TD,Expected}$

where:

$\delta_{TD,Expected}$ = expected relative longitudinal displacement at the rail centroid due to T_D loading per Section 3.6.2.

For most structures, $\delta_{TD,Expected}$ can be approximated by:

$$\delta_{TD,Expected} = \alpha(\Delta T)L_{TU}$$

where:

α = coefficient of thermal expansion for the superstructure

ΔT = 40˚F temperature differential per Section 3.6.2. (ΔT always positive for calculation of $\delta_{TD,Expected}$)

L_{TU} = length of structural thermal unit for a given expansion joint.

For any structure where $\delta_{TD,Expected}$ cannot be approximated with the above equation, $\delta_{TD,Expected}$ shall be verifed by monitoring rail-structure interaction models subject to T_D loading. When a special rail-structure interaction analysis per Section 3.9.7 is required, a detailed temperature analysis shall be required to justify the determination of $\delta_{TD,Expected}$.

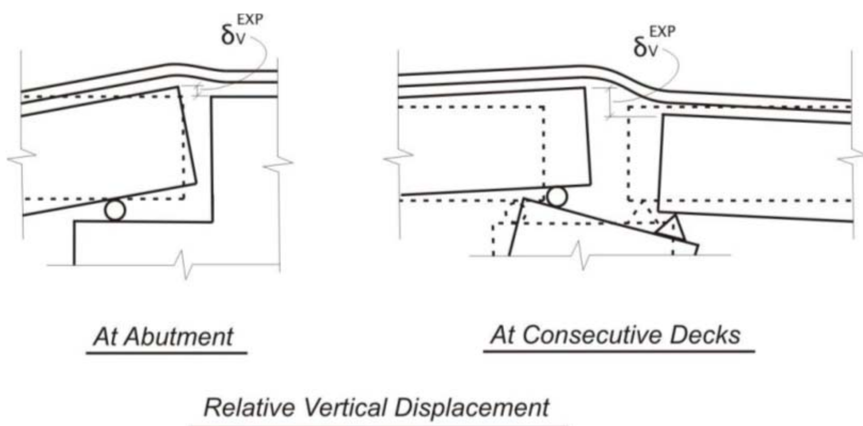
3.6.4 Relative Vertical Displacement at Expansion Joints

The relative vertical displacements (RVD) at structural expansion joints, δ_V^{EXP}, shall be limited in order to control rail bending stress..

The flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

As shown in Figure 3-14, structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

Figure 3-14: Relative Vertical Displacement at Expansion Joints



The RVD at expansion joints (δ_V^{EXP}) shall not exceed the limits given in Table 3-11.

Table 3-11: Relative Vertical Displacement at Expansion Joints Limits

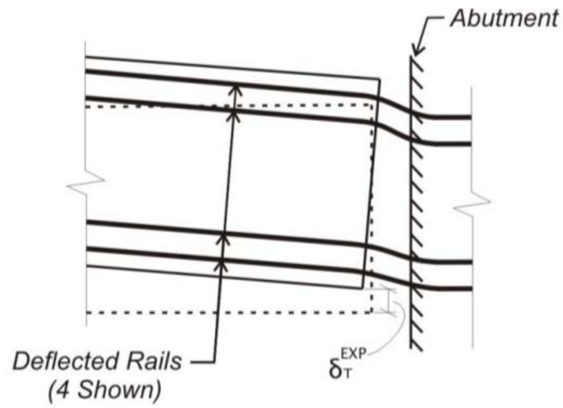
Group	δ_V^{EXP} (inch)	
	Non-ballasted Track	Ballasted Track
4	0.25	0.5
5	0.50	0.75

Refer to Section 3.5.9 for additional RVD limits for track serviceability analysis.

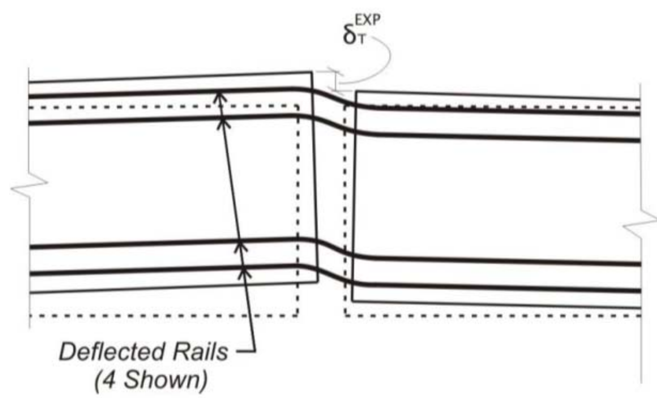
3.6.5 Relative Transverse Displacement at Expansion Joints

The relative transverse displacements (RTD) at structural expansion joints, δ_T^{EXP}, shall be limited in order to prevent excessive rail bending stress. As shown in Figure 3-15, structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

Figure 3-15: Relative Transverse Displacement at Expansion Joints



At Abutment - Plan View



At Consecutive Decks - Plan View

The RTD at expansion joints (δ_T^{EXP}) shall not exceed the limits given in Table 3-12.

Table 3-12: Relative Transverse Displacement at Expansion Joints Limits

Group	δ_T^{EXP} (inch)	
	Non-ballasted Track	Ballasted Track
4	0.08	0.16
5	0.16	0.24

3.6.6 Uplift at Direct Fixation Fasteners for Non-Ballasted Track

For final design, TSI-critical structures with non-ballasted track shall be subject to the following assessment to prevent damage to key track components under Group 4 and Group 5 loading.

For Groups 4 and 5, the direct fixation fastening system capacity, including the anchorage to supporting slab track, shall be designed to withstand calculated uplift force (F_{uplift}) by the factors of safety given in Table 3-13.

Table 3-13: Minimum Factor of Safety for Uplift on Direct Fixation Fasteners

Group	F_{uplift}
4	2.0
5	1.33

Note: Limits apply for non-ballasted track only

Specially designed fasteners with reduced vertical stiffness and/or increased uplift capacity may be required adjacent to structural expansion joints. It shall be demonstrated with proposed preliminary fastener anchorage detailing and correpsonding calculations that the proposed direct fixation fastening system has sufficient capacity for the uplift demand. Assumptions for fastener capacity shall be provided on the plans and supported with calculations.

3.6.7 Permissible Additional Axial Rail Stress Limits

Permissible additional axial rail stress limits were developed considering total allowable rail stresses minus bending stresses due to vertical wheel loads, relative displacements at structural expansion joints, and the initial axial rail stress due to rail temperature and preheat during rail installation (per TM 2.1.5: Track Design).

The permissible additional axial rail stress limits pertain to axial only rail stresses generated by RSI [21].

For rails on the TSI-critical structures and adjacent abutments or at-grade regions, the permissible additional axial rail stresses (σ_{rail}) shall be per Table 3-14.

Table 3-14: Permissible Additional Axial Rail Stress Limits

Group	Range of σ_{rail}	
	Non-ballasted Track	Ballasted Track
4	-14 ksi ≤ σ_{rail} ≤ +14 ksi	-12 ksi ≤ σ_{rail} ≤ +14 ksi
5	-23 ksi ≤ σ_{rail} ≤ +23 ksi	-21 ksi ≤ σ_{rail} ≤ +23 ksi

Note: Compression = Negative (-), Tension = Positive (+)

To approximate the rail bending stress demands, refined models of the fastener/rail assembly at expansion joints were made. These models were subject to vertical and transverse offset (relative) displacements for each side of the expansion joint. At the vertical and transverse limits (see Table 3-11 and Table 3-12), the rail bending stresses were found and subtracted from the total allowable stress to arrive at the permissible additional axial rail stress limits. Refer to the white paper entitled “Rail Stress Evaluation and Fastener Restraint” [21] for more information.

The refined models used the following assumptions:

- Continuous welded rails (AREMA 141RE and EN 60 E1 rail sections) without rail expansion joints, refer to TM 2.1.5: Track Design.
- Non-ballasted and ballasted track types, refer to TM 2.1.5: Track Design
- Rail fasteners with 1.54 kips (6.85 kN) unloaded longitudinal restraint at 27” spacing, see Figure 3-23 and Figure 3-24 for the bi-linear coupling springs on a per foot of track (i.e., two rail) basis.
- Rail fasteners with transverse stiffness of 475 k/ft at 27” spacing.
- Rail fasteners with vertical stiffness in compression of 4610 k/ft per fastener at 27” spacing for nonballasted track and 2360 k/ft per fastener at 27” spacing for ballasted track. In uplift, two conditions were considered for non-ballasted track. For the first condition, uplift stiffness was assumed to be 80 k/ft per fastener from 0” to 0.25” upwards displacement and 1400 k/ft per fastener where displacement exceed 0.25”. For the second non-ballasted condition, a constant uplift stiffness of 115 k/ft per fastener was assumed. For ballasted track, a constant uplift stiffness of 3.2 k/ft per fastener was used.

- Standard rail with a minimum yield strength of 74.0 ksi and an ultimate tensile strength of 142.5 ksi. For project rail type recommendations, refer to TM 2.1.5: Track Design.
- A 330 feet length of structural thermal unit (L_{TU}).
- Straight track or track radius r ≥ 22,000 feet.

3.7 Dynamic Structural Analysis

3.7.1 General

Dynamic structural analysis using actual high-speed trains (LLV) is required in order to determine resonancy induced dynamic impact (I_{LLV}) effects, and limit vertical deck accelerations. Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loading.

For all dynamic structural analysis of high-speed train passage (LLV) the flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered.

To avoid over or underestimating the resonant speeds, two conditions must be investigated:

- Condition #1: lower bound estimate of stiffness and upper bound estimate of mass.
- Condition #2: upper bound estimate of stiffness and lower bound estimate of mass.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 3.9.

Refer to Section 3.2 to determine when dynamic structural analysis of high-speed train passage is required for preliminary and final design.

3.7.2 High Speed Train Loading (LLV)

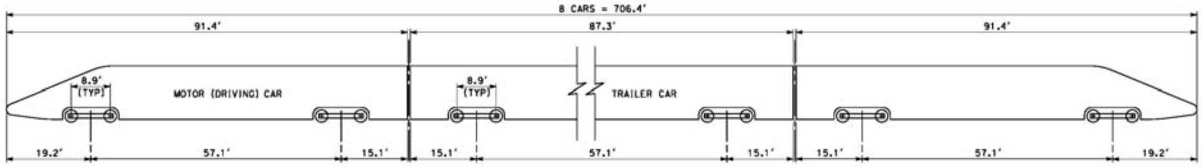
Dynamic structural analysis of high-speed train passage shall consider representative trainsets (LLV), idealized as a series of moving vertical loads at specified axle and truck spacings. Modeling of the train suspension system shall not be required for dynamic structural analysis.

Five trainsets, shown in Figure 3-16 to Figure 3-20 collectively form LLV.

For preliminary design, when a limited dynamic analysis applies per Section 3.2.2, the single trainset shown in Figure 3-16 shall be investigated, subject to selected speeds given in Section 3.7.3.

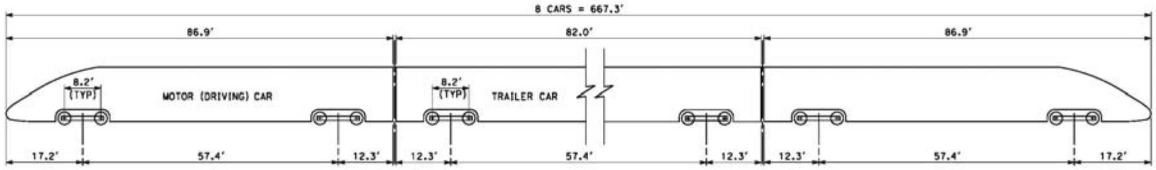
For final design, a full dynamic analysis applies per Section 3.2.3. All five trainsets shown in Figure 3-16 to Figure 3-20 shall be investigated subject to the suite of speeds given in Section 3.7.3.

Figure 3-16: Bombardier-Zerfiro 380 Loading Diagram



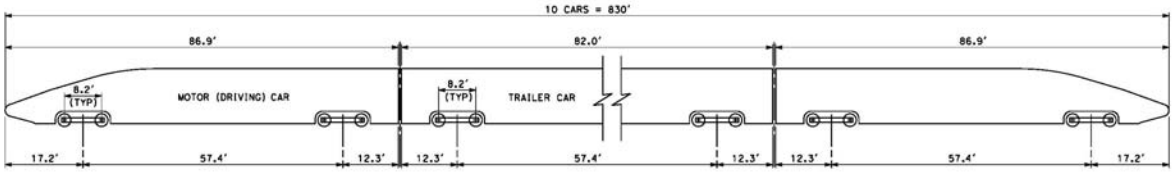
Maximum Axle Load = 18.7 tons Train Weight (Empty) = 509 tons

Figure 3-17: CRH380A Loading Diagram



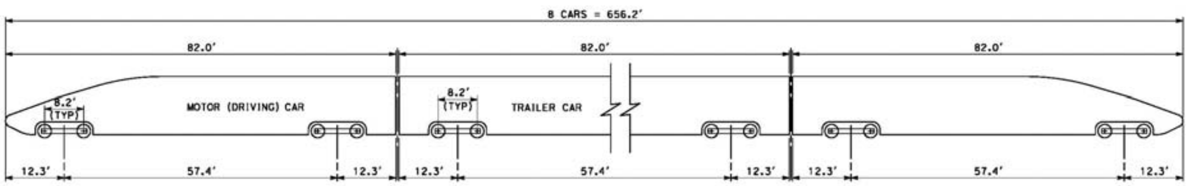
Maximum Axle Load = 16.5 tons Train Weight (Empty) = 529 tons

Figure 3-18: E5 Series Shinkansen Loading Diagram



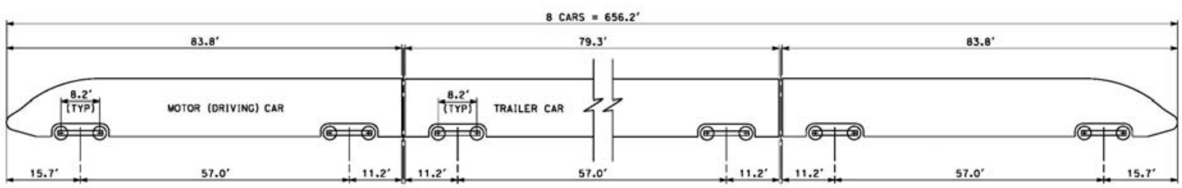
Maximum Axle Load = 12.95 tons Train Weight (Empty) = 500 tons

Figure 3-19: Kawasaki efSET Loading Diagram



Maximum Axle Load = 15.4 tons Train Weight (Empty) = 496 tons

Figure 3-20: Siemens Valero CN Loading Diagram



Maximum Axle Load = 18.7 tons Train Weight (Empty) = 493 tons

3.7.3 Train Speeds

When a limited dynamic structural analysis applies per Section 3.2.2, one trainset shown in Figure 3-17 shall be investigated, subject to following selected speeds:

- The first two resonant speeds.
- Speeds at ±5 mph on each side of the fastest resonant speed.

For final design, a full dynamic structural analysis applies per Section 3.2.3. Each of the five trainsets in Figure 3-17 through Figure 3-20 shall be investigated at the following suite of speeds:

- Speeds from 90 mph up to maximum speed of 1.2 times the line design speed (or 250 mph, whichever is less), by increments of 10 mph [2].
- Smaller increments of 5 mph for ±20 mph on each side of the first two resonant speeds.

Resonant Speeds

For simple spans, resonant speeds [9] may be estimated by:

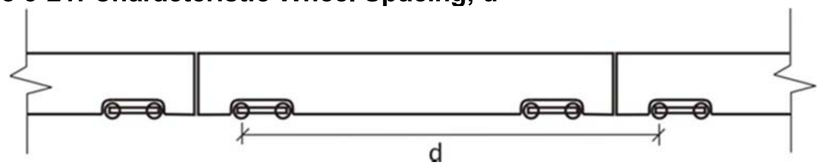
$V_i = n_o d/i$, where V_i = resonant speeds,

n_o = first natural frequency of vertical deflection

d = characteristic wheel spacing, see Figure 3-21

i = resonant mode numbers (e.g., 1, 2, 3, 4, …)

Figure 3-21: Characteristic Wheel Spacing, d



For structures not consisting of simple spans, resonant speeds shall be determined by the dynamic analysis model.

Cancellation Speeds

In addition to resonance, cancellation effects [9] also contribute to the overall dynamic response of elevated structures. For simple spans, cancellation speeds may be estimated by:

$V_i = \dfrac{2\eta_o L}{2i-1}$, where V_i = cancellation speeds,

n_o = first natural frequency of vertical deflection

L = simple span length

i = cancellation mode numbers (e.g., 1, 2, 3, 4, …)

When L/d = 1.5, an optimal design condition exists for which the first mode of resonance aligns with the second mode of cancellation . In this condition, the primary dynamic residual response generated by repeated axle loads can be suppressed. Due to uncertainities associated with the service life of the structure, it may be unrealistic to design a given structure solely for a single characteristic wheel spacing. Nevertheless, optimal span lengths for potential trainsets shall be considered for design.

For non-simple span structures, the interaction between resonant and cancellation speeds may not be readily apparent and shall be investigated by a more detailed dynamic analysis.

3.7.4 Dynamic Vertical Impact Effects

For preliminary design, dynamic vertical impact effects need only be considered when a limited dynamic structural analysis applies per Section 3.2.2. The trainset in Figure 3-16 shall be investigated.

For final design, dynamic vertical impact effects shall be considered for all trainsets in Figure 3-16 through Figure 3-20.

For the high-speed trainsets (LLV), the dynamic model shall be used to determine the dynamic impact effect (I_{LLV}) [2].

In order to determine (I_{LLV}), the maximum dynamic response value, ξ_{dyn}, shall be found for each structural response for single track loading (LLV) over the range of speeds given in Section 3.7.3.

Compared against the corresponding static response value, ξ_{stat}, the dynamic impact effect is:

$$I_{LLV} = \max\left[\frac{\xi_{dyn}}{\xi_{stat}}\right]$$

3.7.5 Vertical Deck Acceleration

Vertical accelerations of TSI-critical structure decks are limited to avoid unsafe wheel-rail contact, and also to minimize passenger discomfort.

When evaluating vertical deck accelerations, an upper bound estimate of stiffness and lower bound estimate of mass, shall be investigated.

Vertical acceleration of TSI-critical structure decks shall be found for single track loading (LLV) over the range of train speeds given in Section 3.7.3. The vertical deck acceleration shall be monitored at the centerline of the loaded track.

The maximum vertical deck acceleration shall be limited to:

- +/- 16.1 ft/s^2 (0.50g) for non-ballasted track [3].
- +/- 16.1 ft/s^2 (0.35g) for ballasted track [3].

Note that this limit pertains to accelerations at the top of structural deck. For acceleration limits within the car body, refer to Section 3.8.4.

3.8 DYNAMIC VEHICLE-TRACK-STRUCTURE INTERACTION ANALYSIS

3.8.1 General

For final design, when TSI-critical structures cannot feasibly meet the requirements in Sections 3.5 through 3.7, a dynamic vehicle-track-structure interaction (VTSI) analysis shall be required as determined as part of conditional approval for the design variance.

Dynamic VTSI also may be required as determined by the Authority for those TSI-critical Complex structures per TM 2.10.4 departing from service-proven concepts, as determined by the Authority during approval of the Track-Structure Interaction Design and Analysis Plan (TSIDAP) per Section 2.4.

When a dynamic VTSI analysis is required, the Contractor shall submit a Vehicle-Track-Structure Interaction Design and Analysis Plan (VTSIDAP) for approval by the Authority. The VTSIDAP shall provide the following detailed information regarding the analysis approach:

- The vehicle models to be used – including mass, stiffness, and damping characteristics of the wheels, trucks, suspension, and body
- The number of trainsets, speeds, and number of cars used for the purpose of analysis
- The approach to be used to generate random track irregularities consistent for the appropriate FRA Track class
- The structural definition, including model boundaries and representation of adjacent earthen embankments and cuts
- The track properties considered, including rail section, fastener, and ballast properties as applicable
- The method used to couple the dynamic train system with the dynamic structure system, including modeling of wheel-rail contact
- The method used to monitor wheel-rail contact forces and carbody accelerations

Additional information for VTSI analysis may be required, as determined by the Authority.

The purpose of VTSI analysis is to verify track safety and passenger comfort by considering the interaction between the vehicle, track, and structure.

Track safety depends primarily upon the contact forces between the rail and the wheel. The ratio of lateral to vertical forces (L/V ratio) is typically used as the primary indicator of derailment. In addition, the magnitudes of lateral and vertical forces imparted by the wheel to the rail must be controlled.

Passenger comfort depends primarily upon the accelerations experienced by passengers within the train car body during travel on and off TSI-critical structures.

3.8.2 Dynamic Vehicle-Track-Structure Interaction Analysis Requirements

For dynamic VTSI, both a dynamic model of the structure and dynamic models of the trainsets shall be used. The coupled interaction of the structure and trainset models shall be considered in either a coupled or iterative method.

Details of structural modeling requirements are given in Section 3.9.

Due to uncertainty of trainset selection, multiple trainset models shall be proposed for dynamic VTSI. Each of the dynamic trainset models shall be consistent with characteristic loading of LLV trainsets as defined in Section 3.7.2, and consider the mass, stiffness, and damping characteristics of the wheels, trucks, suspension, and body.

It is known that vehicle response is highly sensitive to track irregularities. For dynamic VTSI analysis, random track irregularities shall be considered directly within the VTSI model. Random theoretical irregularities shall be developed for FRA Track Classes using a power spectral density function which may be distributed into the time domain by applying the spectral representation method.

Dynamic VTSI analysis shall consider a series of speeds ranging from a minimum of 90 mph up to maximum speed of 1.2 times the line design speed (or 250 mph, whichever is less).

Dynamic VTSI analysis shall consider single track (i.e., one trainset) loading for all structures.

For the dynamic VTSI analysis, a sufficient number of cars shall be used to produce maximum load effects in the longest span of the structure. In addition, a sufficient number of spans within a long viaduct structure shall be considered to initiate any resonance effects in the train suspension.

3.8.3 Dynamic Track Safety Criteria

Dynamic track safety criteria shall not exceed the limits given in Table 3-15 for any trainset across the required speed range.

Table 3-15: Dynamic Track Safety Limits

Parameter	Dynamic Track Safety Criteria
Maximum Single Wheel L/V Ratio	$L/V_{wheel} \leq 0.80$
Maximum Truck Side L/V Ratio	$L/V_{truck\ side} \leq 0.60$
Minimum Single Wheel Dynamic Vertical Load	$V_{wheel,dynamic} \geq 0.15*V_{wheel,static}$
Maximum Net Axle Dynamic Lateral Force	$L_{axle,dynamic} \leq 0.40*V_{axle,static} + 5$ kips

Where:

L/V_{wheel} = Ratio of lateral forces to vertical forces exerted by a single wheel on the rail

$L/V_{truck\ side}$ = Ratio of lateral forces to vertical forces exerted by any one side of a truck on the rail

$V_{wheel,dynamic}$ = Dynamic vertical wheel reaction

$V_{wheel,static}$ = Static vertical wheel load

$L_{axle,dynamic}$ = Dynamic lateral axle reaction

$V_{axle,static}$ = Static vertical axle load

3.8.4 Dynamic Passenger Comfort Criteria

The maximum lateral acceleration within the car body is limited to 1.6 ft/s^2 (0.05 g) for all trainsets and speeds.

The maximum vertical acceleration within the car body is limited to 1.45 ft/s^2 (0.045 g) for all trainsets and speeds.

3.9 MODELING REQUIREMENTS

3.9.1 General

The following modeling requirements for static and dynamic analysis of high-speed train TSI-critical structures are given for project-wide consistency.

3.9.2 Model Geometry and Boundary Conditions

The model shall represent the TSI-critical structure span lengths, vertical and horizontal geometries, column heights, mass and stiffness distribution, bearings, shear keys, column or abutment supports, and foundation conditions.

For isolated TSI-critical structures, with no adjacent structures, the model shall represent the entire structure including abutment support conditions.

For TSI-critical structures with repetitive simply supported spans the model shall have a minimum of twenty (20) spans. Boundary conditions at the ends of the model shall represent the stiffness of any adjacent spans or frames.

For TSI-critical structures with repetitive continuous span frames (i.e., each frame consists of multiple spans with moment transfer between the deck and columns), the model shall have a minimum of five (5) frames. Boundary conditions at the ends of the model shall represent the stiffness of adjacent spans or frames.

Soil springs at the foundations shall be developed based on information provided by the Project Geotechnical Design Report.

For modeling of earthen embankments or cuts at bridge approaches, refer to Section 3.9.9.

3.9.3 Model Stiffness

Structural elements shall be represented by the appropriate sectional properties and material properties.

For frequency analysis, dynamic structural analysis, and dynamic VTSI analysis, both upper and lower bound estimates of stiffness shall be considered.

For track serviceability and RSI analysis, a lower bound estimate of stiffness shall be considered.

For steel superstructure and steel column members, the following shall apply:

- Upper bound stiffness: full steel cross sectional properties, and expected material properties (larger than nominal specified per AASHTO LRFD BDS with California Amendments) shall be used.
- Lower bound stiffness: reduced steel cross sectional properties considering shear lag effects if necessary, and nominal material properties shall be used.

For reinforced, pre-stressed, and post-tensioned concrete superstructure members, the following shall apply:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used. Consideration shall be made for composite action of the superstructure with slab track, and barriers or derailment walls when determining upper bound bending inertias.
- Lower bound stiffness: effective bending inertia, I_{eff}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

For concrete column members, the following shall apply:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used.
- Lower bound stiffness: cracked bending inertia, I_{cr}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

As an alternative to using I_{cr} per CSDC, an effective bending inertia, I_{eff}, which considers the maximum moment demand, M_a, and the cracking moment, M_{cr}, may be used in accordance with AASHTO LRFD BDS with California Amendments. Also, an axially dependent moment-curvature representation of the column stiffness may be used.

3.9.4 Model Mass

For frequency analysis and dynamic analysis using actual high-speed trains, both upper and lower bound estimates of bridge mass shall be considered.

For track serviceability and RSI analysis, an upper bound estimate of bridge mass shall be considered.

For structural dead load (DC) mass, the material unit weights per TM 2.3.2: Structure Design Loads shall be used as the basis for design. For upper bound mass estimate, these unit weights shall be increased by a minimum of 5%. For lower bound mass estimate, these unit weights shall be reduced by a minimum of 5%.

For superimposed dead load (DW), upper and lower bound mass estimates shall be considered.

3.9.5 Model Damping

When performing OBE time history analyses for track serviceability and rail-structure interaction analysis, damping per TM 2.10.4: Seismic Design Criteria shall be used.

When performing dynamic analysis using actual high-speed trains, the peak structural response at resonant speed is highly dependent upon damping. The damping values in Table 3-16 shall be used [2].

Table 3-16: Damping Values for Dynamic Model

Bridge Type	Percent of Critical Damping
Steel and composite	0.5%
Pre-stressed, post-tensioned concrete	1.0%
Reinforced concrete	1.5%

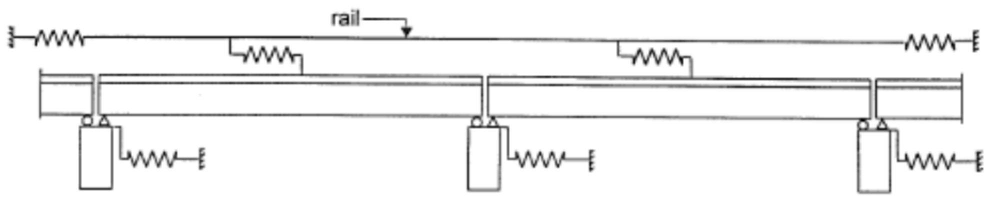
The damping may be increased for shorter spans (< 65 feet), [2]. To justify use of increased damping, the Designer shall provide supporting evidence as part of the PDAP per Section 2.3 or TSIDAP per Section 2.4 as applicable.

When performing dynamic analysis using LLV, soil damping shall be considered in accordance with the Geotechnical Design Report.

3.9.6 Modeling of Rail-Structure Interaction

Longitudinal actions produce longitudinal forces in continuous rails. These forces are distributed to the TSI-critical structures in accordance with the relative stiffness of the track and fasteners, articulation of the structural system, and stiffness of the substructure, refer to Figure 3-22 for a schematic rail-structure interaction model.

Figure 3-22: Rail-Structure Interaction Model



Rail-structure interaction (RSI) may govern the following:

- Location and distance between bridge expansion joints.
- Stiffness of the bridge superstructure.
- Stiffness of the supporting columns and foundations.

RSI shall be performed for all structures using either static or dynamic models. In addition, the model shall, at a minimum, include the axial stiffness of the rails appropriately located upon the superstructure, and longitudinal bi-linear coupling springs between the track and superstructure over the length of the model.

For purposes of this analysis, the continuous welded rail section shall be the EN60 rail per *EN13674-1, Railway applications – Track – Rail – Part 1: Vignole railway rails 46 kg/m and above*. Refer to Table 3-17 for calculated rail section properties to be used for analysis.

The use of the EN60 rail for analysis shall not be construed as a requirement for track design or track construction.

Table 3-17: EN 60 E1 Rail Section Properties

Property	Metric units (given)	US units (calculated)
Mass per meter:	60.21 kg/m	121.4 lb/yd
Cross-sectional area:	76.70 cm^2	11.89 in^2
Moment of inertia x-x axis:	3038.3 cm^4	73.00 in^4
Section modulus – Head:	333.6 cm^3	20.36 in^3
Section modulus – Base:	375.5 cm^3	22.91 in^3
Moment of inertia y-y axis:	512.3 cm^4	12.31 in^4
Section modulus y-y axis:	68.3 cm^3	4.17 in^3

The track type (non-ballasted or ballasted) and corresponding fasteners restraint shall be defined in the PDAP per Section 2.3 or TSIDAP per Section 2.4, as applicable.

Fastener restraint is nonlinear, allowing slippage of the rail relative to the track support structure. Bi-linear coupling springs shall represent non-ballasted track with direct fixation fasteners (refer to Figure 3-23) or ballasted track with concrete ties and elastic fasteners (refer to Figure 3-24) between the rails and superstructure on a per track (i.e., two rail) basis [2]. The non-ballasted relationship represents a pair of fasteners with 1.54 kip (6.85 kN) unloaded longitudinal restraint at 27-inch spacing. The ballasted relationship represents a pair of fasteners on a concrete tie with 1.54 kip (6.85 kN) unloaded longitudinal restraint at 27-inch tie spacing. In each case, the longitudinal restraint is 1.37 k (unloaded) per foot of track and 2.7 k (loaded) per foot of track. The yield displacement varies from 0.02” (non-ballasted) to 0.08” (ballasted).

Figure 3-23: Non-ballasted Track with Direct Fixation Fasteners: Bi-linear Coupling Springs

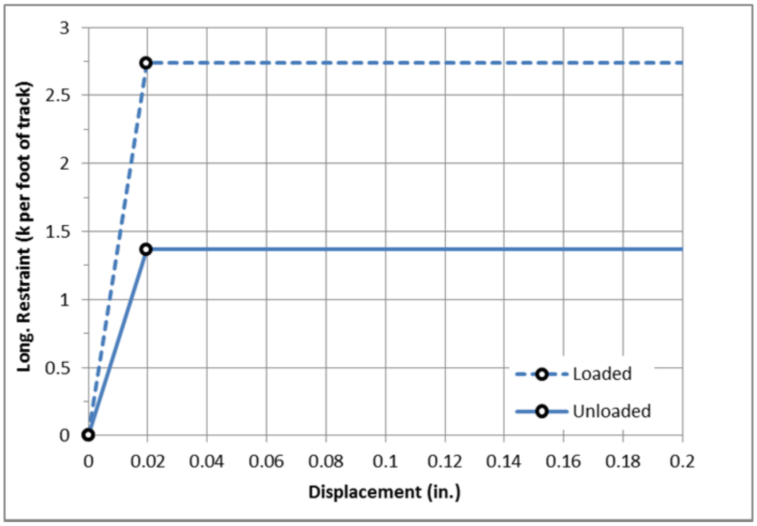
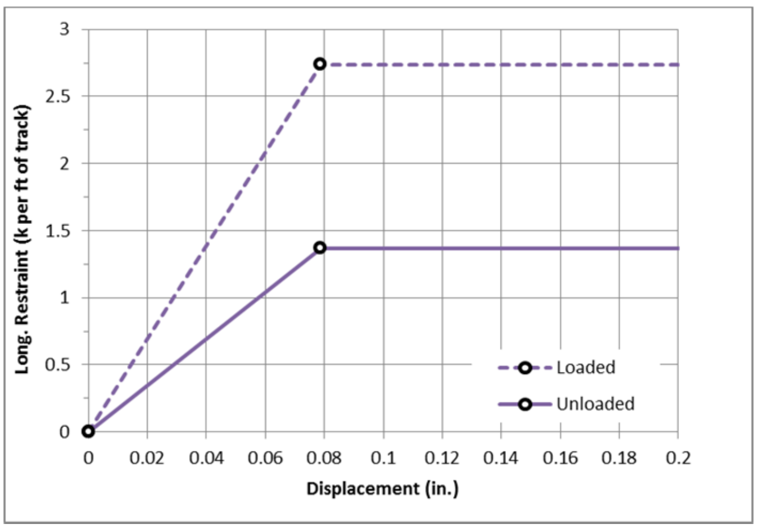


Figure 3-24: Ballasted Track with Concrete Ties and Elastic Fasteners: Bi-linear Coupling Springs



In practice, variations in fastener/tie spacing may be required to accommodate structural expansion joints, deck skew, or other geometric constraints.

Uniform longitudinal restraint shall be verified using the following uniformity criteria:

- Distributed longitudinal restraint calculated for fastener locations over any 10 foot length of track along the structure shall be within +/-20% of the assumed uniform bi-linear coupling relation.

For TSI-critical structures that meet the uniformity criteria, but are designed assuming longitudinal restraints which are not consistent with Figure 3-23 or Figure 3-24, the structure shall be considered to have a nonstandard fastener configuration (NSFC). These structures require an approved design variance and special RSI analysis per Section 3.9.7.

For TSI-critical structures that do not meet the uniformity criteria, the structure shall be considered to have a non-uniform fastener configuration (NUFC). These structures require an approved design variance and a special RSI analysis per Section 3.9.7.

The total number of longitudinal bi-linear coupling springs per each span shall not be less than ten (10) and the spacing between the springs shall not be more than 10 feet.

For vertical and lateral (i.e., transverse) stiffness of fasteners, defined as per foot of track (pair of rails) the following properties shall be used as applicable:

- Non-ballasted track:
  - Vertical stiffness: 4100 k/ft per foot of track
  - Lateral Stiffness: 420 k/ft per foot of track
- Ballasted track:
  - Vertical stiffness: 2100 k/ft per foot of track
  - Lateral Stiffness: 420 k/ft per foot of track

For purposes of evaluating TM 2.10.10 design criteria, constant vertical stiffness shall be used to model fastener compression and tension (uplift).

The assumed fastener stiffness relationships (longitudinal, vertical, and lateral) are to be used for the design of TSI-critical structures only. These relationships provided for RSI models are not to be used for track design. As a means to meet RSI criteria per Section 3.6, the Contractor may propose alternative track solutions (e.g., NSFC, NUFC, Rail Expansion Joints, etc.) through the design variance approval process. The design variance shall be supplemented with a special RSI analysis per Section 3.9.7.

3.9.7 Special Rail-Structure Interaction Analysis

RSI limits in Section 3.6 were developed considering typical fastener configurations on typical structures. For those systems that do not meet these assumptions, new limits shall be developed using a refined analysis.

A special RSI analysis shall be required for those structure and track designs requiring a design variance related to Section 3.6. Specific design variances requiring special RSI analysis include, but are not necessarily limited to: designs requiring nonstandard fastener configurations (NSFC), non-uniform fastener configurations (NUFC), structures with thermal units (L_{TU}) greater than 330 feet, rail expansion joints (REJs).

For final design, the Contractor shall identify and document structure types requiring special RSI analysis as part of the Type Selection process. After completion of Type Selection and upon determination that the selected structure type requires a special RSI analysis, the Contractor shall develop a Rail-Structure Interaction Design and Analysis Plan (RSIDAP) as part of the design variance submittal. The RSIDAP shall formally identify elements requiring special consideration, including but not limited to: refined fastener properties, detailed temperature analysis, refined ballast/nonballasted properties, and rail expansion joint locations. A detailed proposal of analysis procedures used to verify track performance (including track safety, passenger comfort, track maintenance, and rail stress) shall be submitted as part of the RSIDAP.

Examples of special analysis required may include, but are not limited to: development of new RSI limits, development of new analytical model elements, local rail stress modeling, site-specific temperature analysis, analysis of impacts to track maintenance.

3.9.8 Modeling of Rail-Structure Interaction at Model Boundaries

Where an abutment occurs at the ends of TSI-critical structures, the rails and bi-linear coupling springs shall be extended a distance of L_{ext} from the face of the abutment. At the model boundary (i.e., at L_{ext} from abutment), a horizontal boundary spring representing the rail/fastener system behavior shall be used. The boundary spring, which represents unloaded track, shall be elastic-perfectly plastic, with a elastic spring constant of k (in units of kips/feet) yielding at P_b (units in kips), which represents the maximum capacity of an infinite number of elastic fasteners.

The yielding of the boundary spring at P_b is a threshold value that shall be checked throughout the RSI analysis. If at any point during the analysis the boundary spring yields at force P_b, L_{ext}

should be increased and the analysis should be repeated until elastic boundary spring behavior is verified.

The boundary spring behavior depends on the type of track adjacent to the analyzed structure. Values of k, P_b, and L_{ext} are given for non-ballasted and ballasted track types in Table 3-18. Note that the minimum recommended values of L_{ext} are dependent on the average span length of the TSI-critical structures (denoted L_{avg}):

$$L_{avg} = \frac{(L_1 + L_2 + ... + L_n)}{n} = \text{the average span length}$$

Table 3-18: Minimum Recommended Track Extension and Boundary Spring Properties

Non-Ballasted Track (fasteners yield at 0.02 inches) with EN 60 E 1 rail			
Yield Load per foot of non-ballasted track	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.37 kips/ft of track [1.54 kips (6.85 kN) fasteners @ 27" o.c.]	23,800	39.7	$0.1L_{avg} + 325$
Ballasted Track (fasteners yield at 0.08 inches) with EN 60 E 1 rail			
Yield Load per foot of ballasted track	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.37 kips/ft of track [1.54 kips (6.85 kN) fasteners @ 27" o.c.]	11,900	79.5	$0.1L_{avg} + 300$

In the event that an additional bridge or other elevated structure is located within the L_{ext} model boundary distance from the face of an earthen abutment, the additional structure (including all loads and modeling requirements presented in this section) must also be included in the RSI analysis model.

The assumptions used to develop Table 3-18 were expected to apply to majority of TSI-critical structures, which are assumed to be in simply-supported configuration with uniform distribution of fasteners. Refer to the project specific white paper entitled "Model Boundary for Structures with Continuously Welded Rail" [22] for more information. Where a special rail-structure interaction analysis is required per Section 3.9.7, additional investigation shall be required to appropriately define the model boundary.

3.9.9 Modeling of Earthen Embankments or Cuts at Bridge Approaches

Where applicable under RSI Section 3.6.2 Group 4 and Group 5 load cases, the vertical and lateral stiffness of non-ballasted or ballasted track upon earthen embankments or cuts shall be determined to accurately predict relative displacements at abutment expansion joints, and rail stress at the abutment and at-grade regions.

The modeling of earthen embankments or cuts is not required for track serviceability (Section 3.5) or dynamic structural analysis (Section 3.7). However, if the embankment and rails are considered in these models, the vertical and lateral stiffness of non-ballasted or ballasted track upon earthen embankments or cuts shall be determined to accurately predict deformations and accelerations, where applicable.

For special dynamic VTSI analysis the vertical and lateral stiffness of non-ballasted or ballasted track shall be determined to accurately predict the wheel-rail contact forces and carbody accelerations when the vehicle passes through transition zones located between the elevated structure and earthen embankments and cuts.

Vertical stiffness of track upon earthen embankment or cuts shall be developed based upon the specific characteristics of the embankment, cut, or transition structure, as applicable. Guidelines can be found in TM 2.9.10: Geotechnical Analysis and Design Guidelines. Stiffness shall exceed the specified minimum value of 350 pci in accordance with AREMA subgrade requirements [1].

For lateral (i.e., longitudinal and transverse) stiffness of track upon earthen embankments or cuts, consideration of embankment flexibility, non-ballasted track or ballast tie embedment, passive pressure, and friction shall be made in accordance with the Geotechnical Design Report.

OBE ground motions shall be applied concurrently at structural foundations and earthen embankments or cuts to capture the effects between the vibrating structure and the relatively stationary track upon earthen embankment or cut. For tall embankments or specific soil types, lag times and/or amplification effects shall be considered for OBE ground motions in accordance with the Geotechnical Design Report.

3.10 OTHER PENDING ISSUES

3.10.1 General

Other issues will be addressed as the CHST criteria is developed. Among these issues are guidelines for rail breakage.

3.10.2 Guidelines for Rail Breakage

Guidelines for rail breakage are dependent on system capability to identify rail fractures, zero stress temperatures for rail, and longitudinal fastener restraint. Guidelines for these topics are provided in TM 2.1.5: Track Design.

4.0 SUMMARY AND RECOMMENDATIONS

Specific track-structure intearction (TSI) requirements for bridges, aerial structures, grade separations, culverts, and aerial stations supporting high-speed train (HST) tracks have been developed. These requirements encompass TSI-critical structures supporting continuous welded rail upon non-ballasted and ballasted track, without rail expansion joints. The requirements consider track serviceability limits, rail-structure interaction (RSI) limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction (VTSI) analysis limits. To further ensure appropriate structural proportioning for dynamic high-speed train loads, frequency limits are provided for preliminary design and as recommendations for final design.

The requirements concern limiting bridge deformations and accelerations of TSI-critical structures, which can be magnified under high-speed moving trains and lead to numerous issues including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

The CHSTP is the first high-speed train system in the United States with design speeds over 200 mph. The Federal Railway Administration (FRA) has published the final rule [5] addressing vehicle-track interaction safety standards for Class 9 (≥160 mph) track. This technical memorandum uses these revised standards as a basis for allowable structural deformations during track-structure interaction.

This technical memorandum is not intended for tracks supported on grade. Where applicable for RSI analysis, this technical memorandum provides guidance for modeling at-grade earthen embankments or cuts at bridge approaches and abutments.

Based upon TM 2.1.5, rail expansion joints shall be avoided since these are costly to maintain. To accomplish this, the maximum length of a structural thermal unit (i.e., the maximum distance between consecutive fixed points of thermal expansion) shall be 330 feet. In unique circumstances (e.g., long spans), rail expansion joints may be allowed with an approved design variance.

The track-structure interaction analysis framework for both preliminary and final design are given. References are provided to document criteria developmental documents and other pertinent background information regarding track-structure interaction analysis.

5.0 SOURCE INFORMATION AND REFERENCES

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. European Standard EN 1991-2:2003 Traffic Loads on Bridges – Part 2: Traffic Loads on Bridges.
3. European Standard EN 1990:2002 + A1:2005 Basis of Structural Design
4. Taiwan High-Speed Railway Design Manual (2000), Volume 9, Sections 1, 2, 3, and 9
5. Federal Register Vol. 78, No. 49. 49 CFR Parts 213 and 238 – Vehicle/Track Interaction Safety Standards; High Cant Deficiency Operations; Final Rule
6. TM 6.1: Selected Train Technologies
7. ISO 2041. Mechanical vibration, shock and condition monitoring – Vocabulary, 2009
8. Jose M. Goicolea, Felipe Castillo. “Research related to vibrations from high speed railway traffic” Workshop: "Noise and Vibration on High-Speed Railways" Oporto, Portugal 2008.
9. Yang, Yeong-Bin, Zhongda Yao, and Y. S. Wu. Vehicle-bridge Interaction Dynamics: with Applications to High-speed Railways. River Edge, NJ: World Scientific, 2004.
10. Jose M. Goicolea, Felipe Gabaldan, Francisco Riquelme. “Design Issues for Dynamics of High Speed Railway Bridges” International Association for Bridge Maintenance and Safety, Porto 2006.
11. P. Wang, R. Chen, and X. P. Chen. “Dynamic Assessment of Ballastless Track Stiffness and Settlement in High-Speed Railway” ASME Conf. Proc. 2010, 203 (2010)
12. P. Ruge, C. Birk. “Longitudinal forces in continuously welded rails on bridgedecks due to nonlinear track-bridge interaction” Computers & Structures, Volume 85, Issues 7-8, April 2007
13. Martin Lindahl. “Track geometry for high-speed railways” Royal Institute of Technology, Stockholm, Sweden 2001
14. Jose M. Goicolea. “Service limit states for railway bridges in new design codes IAPF and Eurocodes” Technical University of Madrid, Spain
15. José M. Goicolea, Jaime Domínguez, Felipe Gabaldón & Juan A. Navarro. “Resonant effects in short span high speed railway bridges: modelling and design issues” Technical University of Madrid, Spain
16. Don Uzarski. “Introduction to Railroad Track Structural Design” University of Illinois Railroad Engineering Program 2009
17. Editor: Rui Calcada, University of Porto, Portugal. Track-Bridge Interaction on High-Speed Railways: Selected and revised papers from the Workshop on Track-Bridge Interaction on High-Speed Railways, Porto, Portugal, 15–16 October, 2007
18. CHSRP White Paper “Track Serviceability Structural Deformation Limits – Profile”, November 22, 2011.
19. CHSRP White Paper “Track Serviceability Structural Deformation Limits – Alignment”, November 22, 2011
20. CHSRP White Paper “Track Serviceability Structural Deformation Limits – Deck Twist”, November 22, 2011.
21. CHSRP White Paper “Rail Stress Evaluation and Fastener Restraint”, July 17, 2013.
22. CHSRP White Paper “Model Boundary for Structures with Continuously Welded Rail”, July 17, 2013.
23. Professional Standard of the People’s Republic of China, Code for Design of High-Speed Railway (2009), TB 10621-2009, Chapter 7
24. Japanese Standard, 2007.03, Design Standards for Railway Structures and Commentary (Displacement Limits)

6.0 PRELIMINARY DESIGN CRITERIA

6.1 GENERAL

This chapter establishes California High Speed Train Project (CHSTP) requirements for track-structure interaction (TSI) for bridges, aerial structures, grade separations, culverts, and aerial stations supporting high-speed train (HST) tracks. These structures, which are critical to ensuring elevated track performance, are hereafter referred to as "TSI-critical structures".

TSI-critical structures are subject to the following design requirements: structural frequency recommendations, track serviceability limits, rail-structure interaction (RSI) limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction (VTSI) analysis limits.

These requirements are concerned with limiting deformations and accelerations of TSI-critical structures, since the structure response can be dynamically magnified under high-speed moving trains. Excessive deformations and accelerations can lead to unacceptable changes in vertical and horizontal track geometry, excessive rail stress, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

The scope of this section is limited to TSI-critical structures with continuous welded rail (CWR) having no rail expansion joints. Criteria are developed for both non-ballasted and ballasted track forms. The scope is not intended for tracks supported at grade or upon embankments. Where applicable for rail-structure interaction analyses, this technical memorandum provides guidance for modeling at-grade earthen embankments or cuts at bridge approaches and abutments.

Preliminary design level analysis requirements are given in Section 6.5. Table 6-1 summarizes the analysis requirement, including model type, train model/speed, result, and relevant subsections.

Table 6-1:Track-Structure Interaction Analysis Requirements

Analysis Goal	Model Type	Train model	Train speed	Result	Subsection(s)
Frequency Analysis	Dynamic	--	--	Frequency Evaluation	6.7.2, 6.7.3, 6.7.4
Track Serviceability Analysis	Static, For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation Limits	6.8.2 to 6.8.11
Rail-Structure Interaction Analysis	Static (linear or non-linear), For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation And Rail Stress Limits	6.9.2 to 6.9.6
Dynamic Structural Analysis	Dynamic	Single Tracks of Actual High-Speed Train Passage	90 mph to 1.2 Line Speed (or 250 mph whichever is less)	Dynamic Impact Factor, Vertical Deck Acceleration	6.10.2 to 6.10.5

6.2 DESIGN VARIANCES TO TRACK-STRUCTURE INTERACTION DESIGN CRITERIA

Design variances to the track-structure interaction criteria presented in this TM shall be made following the procedure given in TM 1.1.18: Design Variance Guidelines.

Examples of performance criteria variances include:

- Exceedance of allowable deformation limits for the track and structure
- Exceedance of permissible rail stresses, under an OBE event

Examples of operational criteria variances include:

- Temporary closure for repairs following an OBE event
- Extended closures for repairs following a OBE event

- The use of rail expansion joints

Variances to CHSTP performance or operational criteria shall be prepared according to TM 1.1.18.

6.3 PRELIMINARY DESIGN AND ANALYSIS PLAN

For preliminary design, the Designer shall develop and submit to the Authority a Preliminary Design and Analysis Plan (PDAP) for each TSI-critical structure defined as Non-standard or Complex per TM 2.10.4 or as otherwise directed by the Authority.

The PDAP shall provide an overview of the TSI-critical structure, including geometry, design constraints, and key design features. The PDAP shall define the following:

- Track Type (ballasted or non-ballasted track)
- Track Configuration (number of tracks, station components, special track features)
- Maximum Operating Speed and Design Speed
- All thermal unit lengths (LTU), defined as the point of thermal fixity to the next adjacent point of thermal fixity as described in Section 6.6.6.
- Locations and extents for all required alternative track solutions such as non-standard fastener configuration (NSFC), non-uniform fastener configuration (NUFC), or rail expansion joints (REJ) as described in Sections 6.11.6 and 6.11.7.

The preliminary analysis approach for each of the applicable analysis goals in Table 6-1 shall be clearly demonstrated in the PDAP. The analysis approach shall provide a summary of assumptions including, but not necessarily limited to:

- Mass and stiffness variations including assumed structural section properties
- The Track Fastener properties (i.e., longitudinal, vertical, and lateral stiffness)
- Model boundaries for RSI analysis
- The location and magnitude of applied live loads

In addition to issues related to TSI design, the PDAP shall be consistent with the Seismic Design and Analysis Plan (SDAP) required per TM 2.10.4.

6.4 DESIGN REFERENCES AND CODES

Final FRA rules on Vehicle/Track Interaction Safety Standards [5] form the basis for the allowable structural deformations contained within this chapter.

For other issues, this chapter uses guidance drawn from the following design references and codes:

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009 [1]
2. European Standard EN 1991-2:2003 Traffic Loads on Bridges [2]
3. European Standard EN 1990:2002 + A1:2005 Basis of Structural Design [3]
4. Taiwan High Speed Rail (THSR) Corporation Design Specifications [4]
5. People's Republic of China, Code for Design of High-Speed Railway (2009) [18]
6. Japanese Standard 2007.03 - Design Standards for Railway Structures and Commentary (Displacement Limits) [19]

6.5 Analysis Requirements

6.5.1 General

As part of LRFD force-based design, static analysis is required for load combinations including Cooper E-50 maintenance and construction trains (LLRR), actual high-speed train loads (LLV), and vertical impact effects (I and I_{LLV}). Refer to TM 2.3.2: Structural Design Loads.

Additional analyses are required beyond what is provided for in this technical memorandum in order to determine basic structural proportioning, ensure track safety, and provide passenger comfort for high-speed train operation on TSI-critical structures.

6.5.2 Preliminary Design Requirements

Frequency analysis, track serviceability analysis, and rail-structure interaction (RSI) analysis shall apply for preliminary design of TSI-critical structures.

For preliminary design, where a TSI-critical structure falls above the recommended lower bound frequency thresholds (Section 6.7), the following applies:

- For Standard, Non-Standard and Complex structures per TM 2.10.4, dynamic structural analysis of high-speed train passage shall not be required.

For preliminary design, where a TSI-critical structure falls below the recommended lower bound frequency thresholds (Section 6.7), the following applies:

- For Standard, Non-Standard, and Complex structures per TM 2.10.4, a limited dynamic structural analysis of high-speed train passage per Section 6.10.2 at selected speeds per Section 6.10.3 shall be required.

6.6 Design Parameters

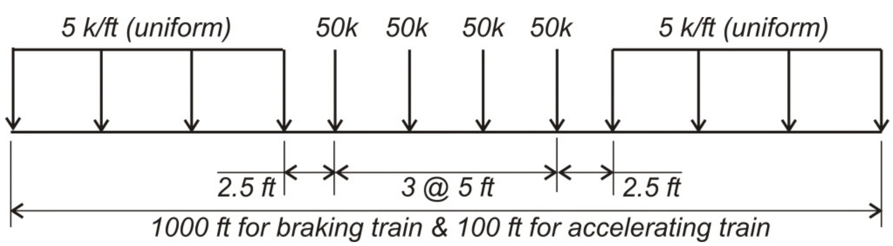
6.6.1 General

The following defines loading to be used for track serviceability and rail-structure interaction analysis.

6.6.2 Modified Cooper E-50 Loading (LLRM)

Modified Cooper E-50 loading (LLRM) per Figure 6-1 shall be used for track serviceability analysis per Section 6.8, and rail-structure interaction analysis per Section 6.9. LLRM loading is on a per track (i.e., two rail) basis.

Figure 6-1: LLRM Loading



6.6.3 Vertical Impact Effect (I)

The vertical impact effect (I) used with Modified Cooper E-50 loading (LLRM) shall be vertical impact effect from LLRR per TM 2.3.2: Structure Design Loads.

Dynamic vertical impact effects (I_{LLV}) caused by high-speed trainsets (LLV) shall be found per Section 6.10.4.

6.6.4 Centrifugal Force (CF)

The centrifugal force (CF) used with Modified Cooper E-50 loading (LLRM) shall be determined per TM 2.3.2: Structure Design Loads. The maximum CF calculated for LLRR and LLV shall be used, whichever governs.

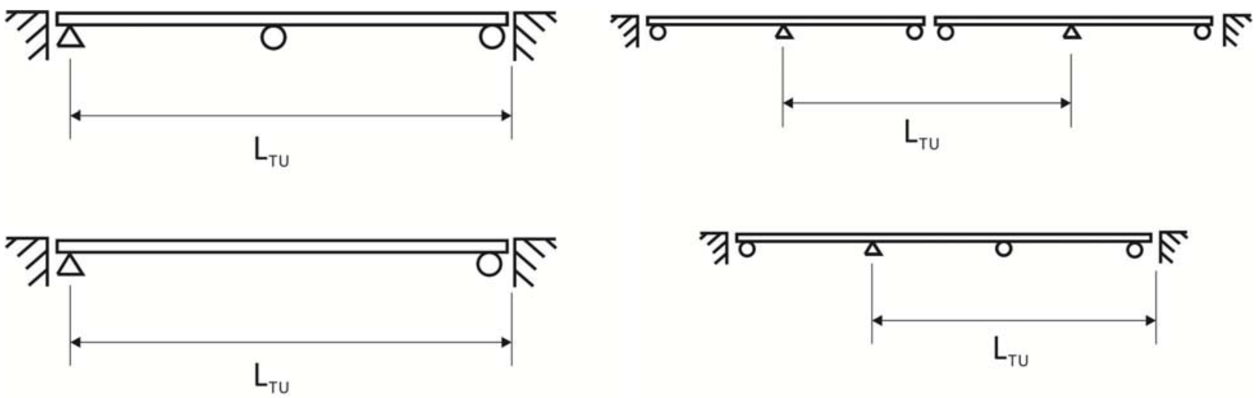
6.6.5 Accelerating and Braking Force (LF)

The longitudinal accelerating and bracking forces (LF) used with Modified Cooper E-50 loading (LLRM) shall be determined using the approach for LLV loading per TM 2.3.2: Structure Design Loads.

6.6.6 Structural Thermal Unit

Per TM 2.1.5: Track Design, rail expansion joints shall be avoided since these are costly to maintain. To accomplish this, the maximum length of a structural thermal unit (L_{TU}), defined as the distance between consecutive fixed points of thermal expansion, shall be 330 feet. Refer to Figure 6-2.

Figure 6-2: Structural Thermal Unit



In unique circumstances (e.g., long spans), rail expansion joints may be allowed with an approved design variance and the additional requirement of special rail-structure interaction analysis per Section 6.11.7.

6.7 FREQUENCY ANALYSIS

6.7.1 General

Frequency limits are placed on the fundamental mode shapes of TSI-critical structures, in order to ensure well-proportioned structures and minimize resonancy effects.

Upper and lower bound mass and stiffness assumptions shall be evaluated per the modeling requirements as given in Section 6.11.

6.7.2 Recommended Range of Vertical Frequency of Span

The recommended vertical lower bound frequency threshold is known to favorably resist high-speed train resonance actions. It is recommended that structures be proportioned to fall above this lower bound threshold.

Where a structure falls below the recommended vertical frequency threshold, then additional analysis shall be required per Section 6.5.

Vertical frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For vertical frequency analysis, two conditions must be investigated:

- Condition #1: a lower bound estimate of stiffness and upper bound estimate of mass

- Condition #2: an upper bound estimate of stiffness and lower bound estimate of mass

Condition #1 will govern the lower bound threshold. Condition #2 is required for future structural assessment.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 6.11.

The recommended threshold for the first natural frequency of vertical deflection, η_{vert} [Hz], primarily due to bending of the span is the following:

$$\eta_{\text{vert}} \geq \eta_{\text{lower}}$$

Where:

$\eta_{lower} = 313.09L^{-0.917}$ for $L \leq 330$ feet

where L = effective length of span (feet)

For simple spans, L shall be the span length.

For continuous spans, L shall be the following:

$$L = k\left(L_{\text{average}}\right)$$

Where:

$$L_{\text{average}} = \frac{(L_1 + L_2 + ... + L_n)}{n}$$ = the average span length

$n =$ the number of spans

$$k = \left(1 + \frac{n}{10}\right) \leq 1.5$$

For portal frames and closed frame bridges, L shall be:

- Single span: consider as three continuous spans, with the first and third span being the vertical length of the columns, and the second span the girder length.
- Multiple spans: consider as multiple spans, with the first and last span as the vertical length of the end columns, and the interior spans the girder lengths.

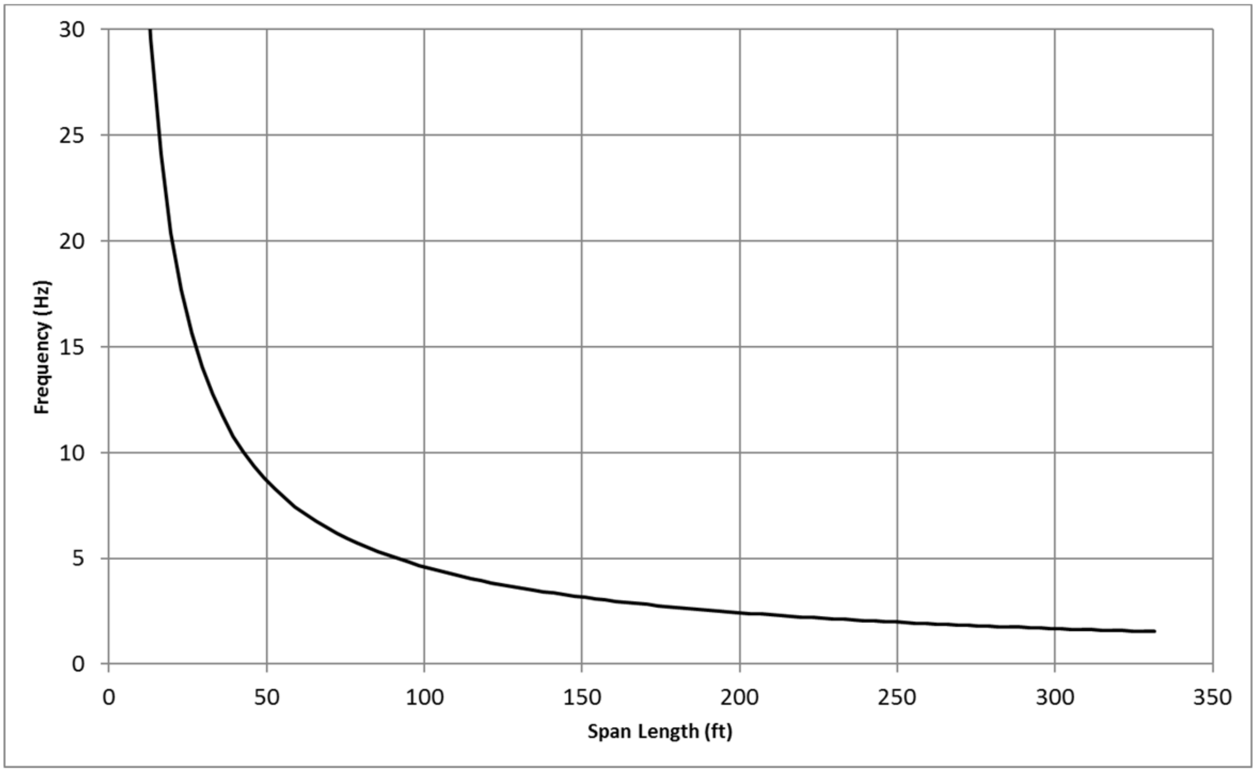
For spans with end diaphragms at abutments (fixed supports at abutments), the following L shall apply:

- Single span, fixed at one abutment: consider as two continuous spans, with the first span equal to 0.05 times the girder length, and the second span the girder length.
- Single span, fixed at both abutments: consider as three continuous spans, with the first and the third span equal to 0.05 times the girder length, and the second span the girder length.
- Multiple spans, fixed at one abutment: consider as multiple spans, with the first span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.
- Multiple spans, fixed at both abutments: consider as multiple spans, with the first and last span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.

For single arch, archrib, or stiffened girders of bowstrings, L shall be the half span.

Refer to Figure 6-3 for the recommended lower bound threshold of vertical frequency.

Figure 6-3: Recommended Lower Bound Threshold of Vertical Frequency



6.7.3 Recommended Lower Bound Torsional Frequency of Span

Recommendations for lower bound torsional frequency are to proportion structures to favorably resist high-speed train actions.

All torsional frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For torsional frequency analysis, two conditions must be investigated, consistent with vertical frequency analysis:

- Condition #1 – a lower bound estimate of stiffness and upper bound estimate of mass
- Condition #2 – an upper bound estimate of stiffness and lower bound estimate of mass

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 6.11.

For Conditions #1 and #2, the first torsional frequency, $\eta_{torsion}$, of the span shall be greater than 1.2 times the corresponding first natural frequency of vertical deflection, η_{vert} [2].

6.7.4 Recommended Lower Bound Transverse Frequency of Span

Recommendations for the lower bound transverse frequency favorably resists high-speed train actions.

For transverse frequency analysis, two conditions shall be investigated:

- Condition #1 – consideration of flexibility of superstructure only, excluding the flexibility of bearings, columns, and foundations, assuming supports at ends of the span are rigid [8].
- Condition #2 – consideration of flexibility of superstructure and substructure, including flexibility of bearings, columns, shear keys, and foundations.

For transverse frequency analysis, a lower bound estimate of stiffness and upper bound estimate of mass shall be used, refer to Section 6.11.

For Condition #1, the first natural frequency of transverse deflection, η_{trans}, of the span shall not be less than 1.2 Hz [3].

For Condition #2, no frequency recommendation is provided, but shall be recorded for future structural assessment.

6.8 TRACK SERVICEABILITY ANALYSIS

6.8.1 General

Track serviceability analysis, using modified Cooper E-50 loading, provides limits to allowable structural deformations. These track serviceability limits are developed for structures supporting continuous welded rail without rail expansion joints.

Deformation limits are developed for limit states based on maintenance, passenger comfort, and track safety requirements.

For track serviceability analysis, the flexibility of superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered.

In order to avoid underestimating deformations, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Details of modeling requirements are given in Section 6.11.

6.8.2 Track Serviceability Load Cases

Track serviceability loads cases shall include [4] :

- Group 1a: $(LLRM + I)_1 + CF_1 + WA$
- Group 1b: $(LLRM + I)_2 + CF_2 + WA$
- Group 1c: $(LLRM + I)_m + CF_m + WA$
- Group 2: $(LLRM + I)_1 + CF_1 + WA + WS + WL_1$
- Group 3: $(LLRM + I)_1 + CF_1 + OBE$

where:

$(LLRM + I)_1$ = one track of (LLRM + I) plus impact

$(LLRM + I)_2$ = two tracks of (LLRM + I) plus impact

$(LLRM + I)_m$ = multiple tracks per Section 6.8.5 of (LLRM + I) plus impact

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (one track) per TM 2.3.2: Structure Design Loads

CF_2 = centrifugal force (two tracks) per TM 2.3.2: Structure Design Loads

CF_m = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

WA = water loads (stream flow) per TM 2.3.2: Structure Design Loads

WS & WL_1 = wind on structure and wind on one 1000’ LLRM train per TM 2.3.2: Structure Design Loads

OBE = Operating Basis Earthquake per TM 2.10.4: Seismic Design Criteria

Note that Group 1c is used for Section 6.8.5 only.

Static analysis and linear superposition of results shall beallowed for Groups 1b, 1c, and 2.

For determining OBE demands in Group 3, equivalent static analysis, dynamic response spectum, or time history (linear or non-linear) analysis may be used, in accordance with the approved PDAP per Section 6.3 and TM 2.10.4: Seismic Design Criteria as applicable. Refer to TM 2.10.4: Seismic Design Criteria for additional OBE modeling requirements.

For track serviceability analysis, non-linear track-structure interaction modeling (refer to Section 6.11.6) is not required, but may be used. For Group 3, superposition of static (i.e., $(LLRM + I)_1 + CF_1$) and either static or dynamic OBE shall be allowed.

6.8.3 Vertical Deflection Limits: Group 1a

Vertical deflection limits for Group 1a are to address maintenance, passenger comfort, and track safety issues.

For Group 1a, the maximum static vertical deck deflection (max Δ_{1a}), in the most unfavorable position, shall not exceed the limits given in Table 6-2.

Table 6-2: Vertical Deflection Limits: Group 1a

Limit	Span Length				
	L ≤ 125ft	L=175ft	L=225ft	L=275ft	L≥330ft
max Δ_{1a}	L/3500	L/3180	L/2870	L/2550	L/2200

Note: Limits apply for both non-ballasted and ballasted track

For span lengths not explicitly referenced in Table 6-2, use linear interpolation.

6.8.4 Vertical Deflection Limits: Group 1b

Vertical deflection limits for Group 1b are to address maintenance, passenger comfort, and track safety issues.

For Group 1b, the maximum static vertical deck deflection (max Δ_{1b}), in the most unfavorable position, shall not exceed the limits given in Table 6-3.

Table 6-3: Vertical Deflection Limits: Group 1b

Limit	Span Length				
	L ≤ 125ft	L=175ft	L=225ft	L=275ft	L≥330ft
max Δ_{1b}	L/2400	L/2090	L/1770	L/1450	L/1100

Note: Limits apply for both non-ballasted and ballasted track

For span lengths not explicitly referenced in Table 6-3, use linear interpolation.

6.8.5 Vertical Deflection Limits: Group 1c

Vertical deflection limits for Group 1c are to provide practical guidance for structures containing three or more tracks operating at speeds less than 90 mph. This guidance is consistent with established European codes.

$(LLRM + I)_m$ and CF_m loading shall be applied in a manner consistent with the case of multiple tracks on structures as described below:

- For 2 tracks, full live load on 2 tracks.
- For 3 tracks, full live load on 2 tracks and one-half on the other track.
- For 4 tracks, full live load on 2 tracks, one-half on one track, and one-quarter on the remaining one.
- For more than 4 tracks, to be considered on an individual basis.

The tracks selected for loading shall be those tracks which will produce the most critical design condition on the member under consideration.

For Group 1c, where the structures support three or more tracks, the maximum static vertical deck deflection (max Δ_{1c}), in the most unfavorable position, shall not exceed L/600 for all span lengths [3]. This limit applies for both non-ballasted and ballasted track.

In the event that structures support 3 or more tracks, and 3 or more trains can be anticipated to be on the same structure at speeds greater than 90 mph, limits defined for Group 1b shall apply. For these structures, representative live load conditions must be developed on a case-by-case basis.

6.8.6 Transverse Deflection Limits

Transverse deflection limits are to address maintenance, passenger comfort, and track safety issues.

The transverse deflection within the span (Δ_{trans}), shown in Figure 6-4, shall not exceed the limits given in Table 6-4.

Figure 6-4: Transverse Span Deformation Limits

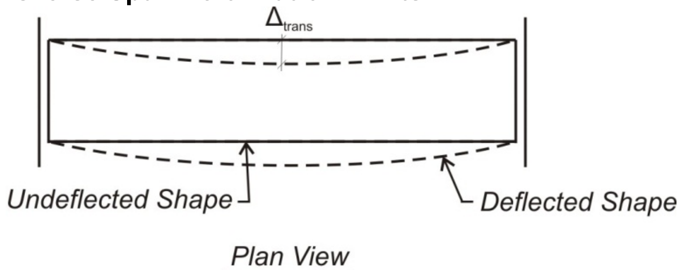


Table 6-4: Transverse Deflection Limits

Group	Δ_{trans} (feet)
1a	L^2/(864,800)
1b	L^2/(447,200)
2	L^2/(276,800)
3	L^2/(276,800)

Note: Limits apply for both non-ballasted and ballasted track

6.8.7 Rotation about Transverse Axis Limits

Rotation about transverse axis limits are to control excessive rail axial and bending stress, provide traffic safety (i.e., guard against wheel unloading due to abrupt angular changes in track geometry), and provide passenger comfort.

Due to rotation about the transverse axis, imposed axial rail displacement is a linear function of the distance between the rail centroid and top of the bridge bearings. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about transverse axis at deck ends (θ_t), shown in Figure 6-5, shall be defined by the following equations:

$\theta_t = \theta$, for abutment condition

$\theta_t = \theta_1 + \theta_2$, between consecutive decks

Also, the maximum relative axial displacement at the rail centroid (δ_t) due to rotation about transverse axis, shown in Figure 6-5, shall also be defined by the following equations:

$\delta_t = \theta h$, for abutment condition

$\delta_t = \delta_1 + \delta_2 = \theta_1 h_1 + \theta_2 h_2$, between consecutive decks.

where:

θ_t (radians): total rotation about transverse axis, see Table 6-5

δ_t (in): total relative displacement at the rail centroid, see Table 6-5

θ (radians): rotation of the bridge bearing at abutment

θ_1 (radians): rotation of the first bridge bearing

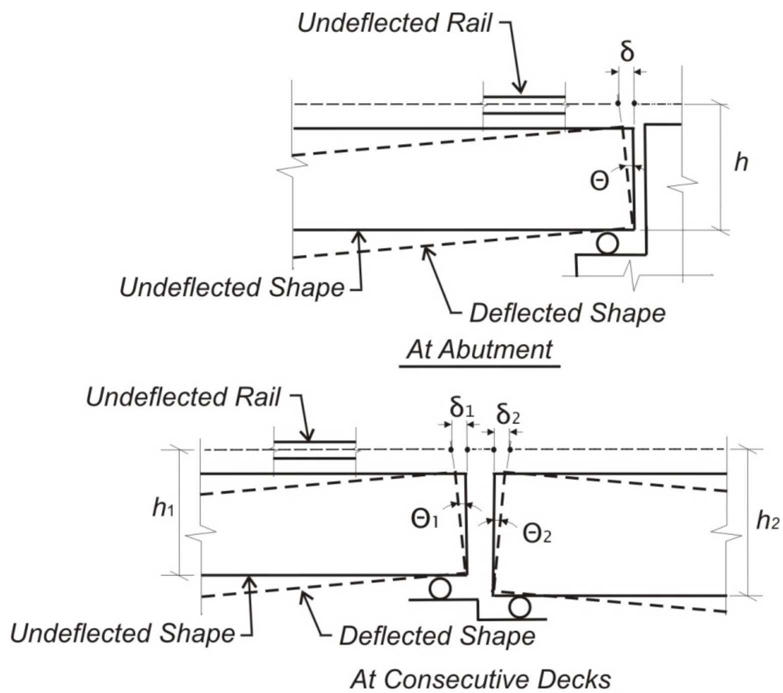
θ_2 (radians): rotation of the second bridge bearing

h (in): the distance between the rail centroid and the bridge bearing at abutment

h_1 (in): the distance between the rail centroid and the top of the first bridge bearing

h_2 (in): the distance between the rail centroid and the top of the second bridge bearing

Figure 6-5: Rotation about Transverse Axis at Deck Ends



The total rotation about transverse axis (θ_t) and the total relative displacement at the rail centroid (δ_t) shall not exceed the limits given in Table 6-5.

Table 6-5: Rotation about Transverse Axis and Relative Displacement at Level of the Rail Limits

Group	θ_t (radians)	δ_t (inches)	
		Non-ballasted Track	Ballasted Track
1a	0.0012	0.33	0.33
1b	0.0017	0.33	0.33
2	0.0026	0.67	0.67
3	0.0026	0.67	0.67

6.8.8 Rotation about Vertical Axis Limits

Rotation about vertical axis limits are to control rail axial and bending stress, provide track safety, and provide passenger comfort by limiting changes in horizontal track geometry at bridge deck ends.

Due to rotation about the vertical axis, imposed longitudinal rail displacement is a linear function of the distance between the centerline of span and the outermost rail. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about vertical axis at deck ends (θ_v), shown in Figure 6-6, shall be defined by the following equations:

$\theta_v = \theta$, for abutment condition

$\theta_v = \theta_A + \theta_B$, between consecutive decks

Also, the maximum axial displacement at the outermost rail centroid (δ_v) due to rotation about vertical axis, shown in Figure 6-7, shall be defined by the following equations:

$\delta_v = \theta w$, for abutment condition

$\delta_v = \delta_A + \delta_B = \theta_A w_A + \theta_B w_B$, between consecutive decks.

where:

θ_v (radians): total rotation about vertical axis, see Table 6-6

θ (radians): rotation of the bridge at abutment

θ_A (radians): rotation of the first span

θ_B (radians): rotation of the second span

w (in): the distance between the centerline span and outermost rail centroid at abutment

w_A (in): the distance between the centerline span and outermost rail centroid of first span

w_B (in): the distance between the centerline span and outermost rail centroid of second span

δ_v (in): total relative displacement at the outermost rail centroid, see Table 6-6

δ (in): relative displacement at the outermost rail centroid, at abutment

δ_A (in): relative displacement at the outermost rail centroid, first span

δ_B (in): relative displacement at the outermost rail centroid, second span

Figure 6-6: Rotation about Vertical Axis at Deck Ends – Global View

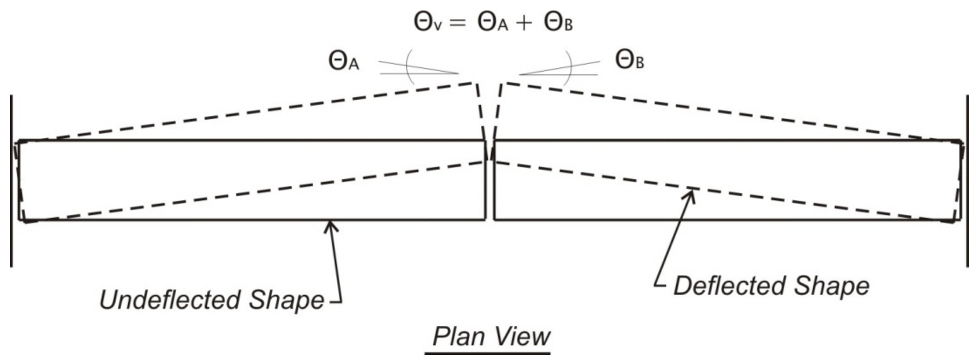
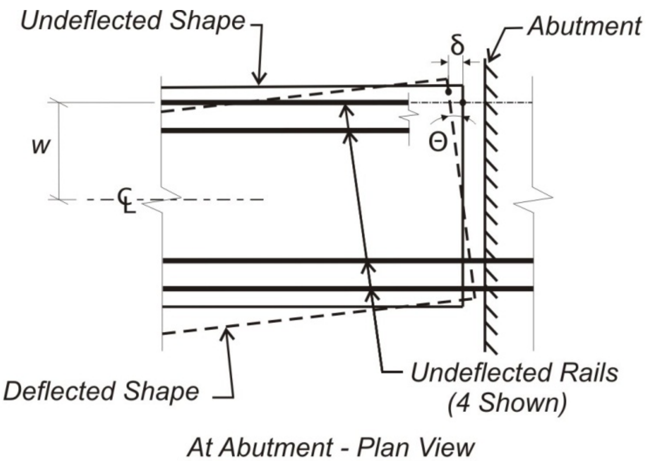
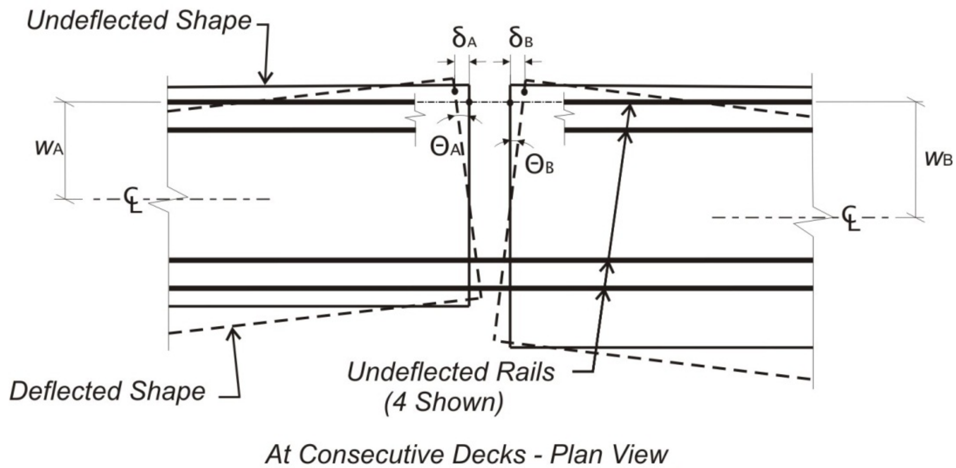


Figure 6-7: Rotation about Vertical Axis at Deck Ends – Local View





The total rotation about vertical axis (θ_v) and the total relative displacement at the outermost rail centroid (δ_v) shall not exceed the limits given in Table 6-6.

Table 6-6: Rotation about Vertical Axis and Relative Displacement at Outermost Rail Limits

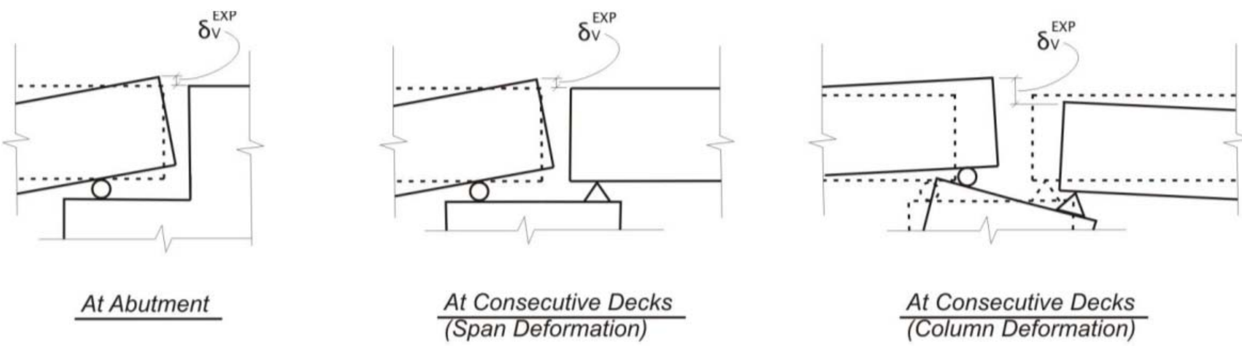
Group	θ_v (radians)	δ_v (inches)	
		Non-ballasted Track	Ballasted Track
1a	0.0007	0.33	0.33
1b	0.0010	0.33	0.33
2	0.0021	0.67	0.67
3	0.0021	0.67	0.67

6.8.9 Relative Vertical Displacement at Expansion Joints – Track Serviceability

Relative vertical displacements (RVD) at structural expansion joints, δ_v^{EXP}, are limited in order to ensure track safety subject to deck end rotation and vertical bearing deformation. As shown in Figure 6-8 structural expansion joints between adjacent deck ends, and between deck ends and abutments, shall be considered.

The flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

Figure 6-8: Relative Vertical Displacement at Expansion Joints – Track Serviceability



The RVD at expansion joints (δ_v^{EXP}), shall not exceed the limits given in Table 6-7.

Table 6-7: Relative Vertical Displacement at Expansion Joints Limits – Track Serviceability

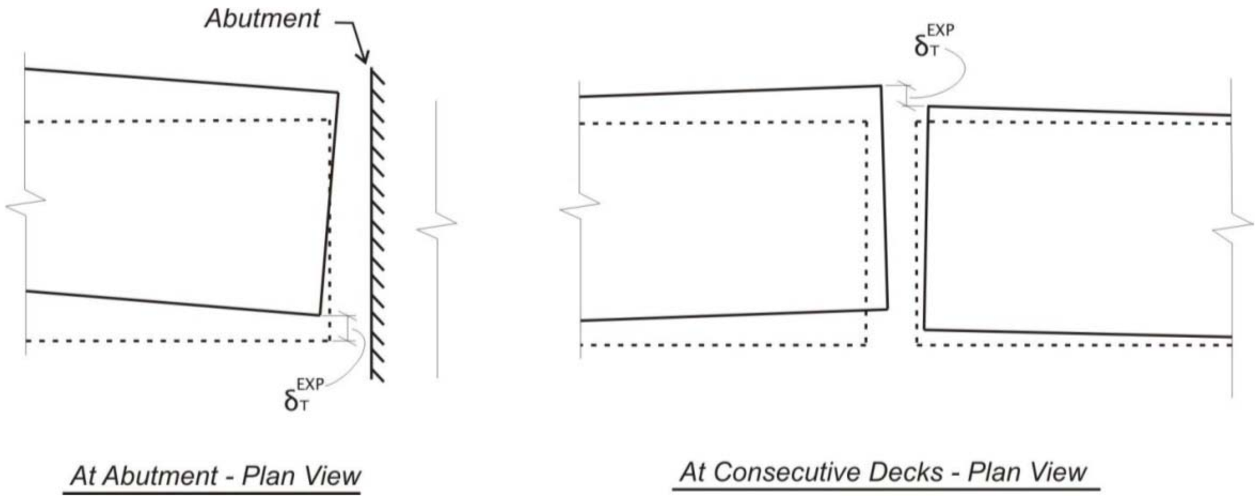
Group	δ_v^{EXP} (inches)
1a	0.25
1b	0.25
2	-
3	-

Note: Limits apply for both non-ballasted and ballasted track

6.8.10 Relative Transverse Displacement at Expansion Joints – Track Serviceability

Relative transverse displacements (RTD) at structural expansion joints, δ_T^{EXP}, are limited in order to ensure track safety subject to shear key and lateral bearing deformation. As shown in Figure 6-9, structural expansion joints between adjacent deck ends, and between deck ends and abutments, shall be considered.

Figure 6-9: Relative Transverse Displacement at Expansion Joints – Track Serviceability



The RTD at expansion joints (δ_T^{EXP}), shall not exceed the limits given in Table 6-8.

Table 6-8: Relative Transverse Displacement at Expansion Joints Limits – Track Serviceability

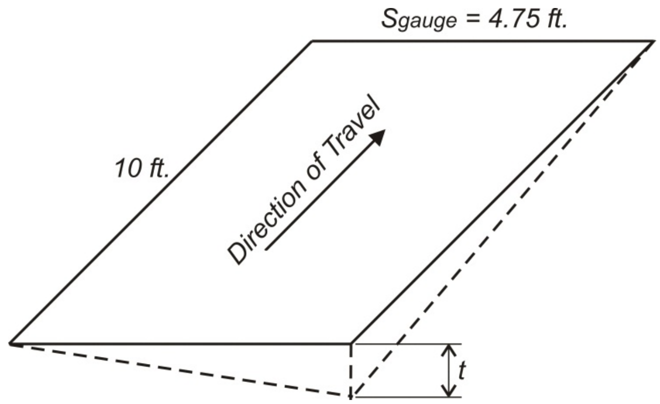
Group	δ_T^{EXP} (inches)
1a	0.08
1b	0.08
2	-
3	-

Note: Limits apply for both non-ballasted and ballasted track

6.8.11 Deck Twist Limits

The deck twist, t, is defined as the relative vertical deck displacment of a given bogie contact point from a plane defined by the remaining three bogie contact points on a track gauge of 4.75 feet over a bogie length of 10 feet, refer to Figure 6-10. Deck twist limits ensure that the four wheel contact points of a bogie are not too far from a plane.

Figure 6-10: Deck Twist Diagram



Maximum deck twist (t_{max}) below tracks shall not exceed the limits given in Table 6-9.

Table 6-9: Deck Twist Limits

Group	t_{max} (in/10 ft)
1a	0.06
1b	0.06
2	0.17
3	0.17

Note: Limits apply for both non-ballasted and ballasted track

6.9 RAIL-STRUCTURE INTERACTION ANALYSIS

6.9.1 General

Rail-structure interaction (RSI) analysis, using modified Cooper E-50 loading (LLRM), shall be used to limit relative longitudinal, vertical, and transverse displacements at structural expansion joints, and limit axial rail stress in order to minimize the probability of rail fracture. Deformation and rail stress limits were developed considering the accumulation of displacement demands and rail bending stresses under the controlling load combinations.

Details of RSI modeling requirements are given in Section 6.11.6.

For RSI analysis, the flexibility of superstructure, bearings, shear keys, columns, and foundations shall be considered.

For all RSI analysis, in order to avoid underestimating deformations and rail stress, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Limits on expansion joint displacement, fastener performance, and rail stress are provided in Sections 6.9.3 through 6.9.6. These limits only apply if all assumptions and modeling requirements given in Section 6.11.6 are valid. For structures requiring alternative assumptions or modeling techniques, an approved design variance and a special RSI analysis per Section 6.11.7 shall be required.

Deformation limits and rail stress limits were developed considering the accumulation of displacement demands and rail bending stresses under the controlling load combinations.

Refer to Section 6.5 to determine when rail-structure interaction analysis is required at preliminary design.

6.9.2 Rail-Structure Interaction Load Cases

Rail-structure interaction (RSI) load cases include [4]:

- Group 4: $(LLRM + I)_2 + LF_2 \pm T_D$
- Group 5: $(LLRM + I)_1 + LF_1 \pm 0.5T_D + OBE$

where:

$(LLRM + I)_1$ = single track of Modified Cooper E-50 (LLRM) plus vertical impact effect

$(LLRM + I)_2$ = two tracks of Modified Cooper E-50 (LLRM) plus vertical impact effect

I = vertical impact effect from LLRR per TM 2.3.2: Structure Design Loads

LF_1 = braking forces (apply braking to one track) for LLV loading per TM 2.3.2: Structure Design Loads

LF_2 = braking and acceleration forces (apply braking to one track, acceleration to the other track) for LLV loading per TM 2.3.2: Structure Design Loads

T_D = temperature differential of ±40˚F between rails and deck, applied to the superstructure.

OBE = Operating Basis Earthquake per TM 2.10.4: Seismic Design Criteria

Groups 4 and 5 are to provide relative longitudinal, vertical, and transverse displacement limits at expansion joints, and design for uplift at direct fixation rail. Groups 4 and 5 are also used to limit rail stress, accounting for thermal effects (i.e. $\pm T_D$).

Modeling of nonlinear RSI effects, as given in Section 6.11.6, shall be required to give realistic demands. Experience has shown that linear modeling of RSI is overly conservative.

For Group 5, non-linear time-history OBE analysis (i.e., non-linear RSI) shall be used for design. $(LLRM + I)_1 + LF_1$ may be idealized as a set of stationary load vectors placed upon the structure in the most unfavorable position. Refer to TM 2.10.4: Seismic Design Criteria for additional OBE modeling requirements.

6.9.3 Relative Longitudinal Displacement at Expansion Joints

Relative longitudinal displacements (RLD) at structural expansion joints, δ_L^{EXP}, are limited in order to prevent excessive rail axial stress. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

RLD at structural expansion joints, δ_L^{EXP}, has components due to both structural translation and structural rotation. For structural rotation, RLD is a function of distance from center of structure rotation to rail centroid. Therefore, δ_L^{EXP} shall be monitored relative to the original rail centroid location, and consist of structural movement alone.

δ_L^{EXP}, determined at the rail centroid, consists of separate components:

- δ_{LF} = component due to acceleration and braking only, refer to Figure 6-11.
- δ_{LLRM+I} = component due to vertical train plus impact loads only, refer to Figure 6-12.
- δ_{OBE} = component due to OBE only (refer to Figure 6-13), comprised of:
    - o $\delta_{OBE(L)}$ = longitudinal displacement subcomponent due to OBE.
    - o $\delta_{OBE(V)}$ = rotation about vertical axis subcomponent due to OBE.
    - o $\delta_{OBE(T)}$ = rotation about transverse axis subcomponent due to OBE.
    - o $\delta_{OBE} = \delta_{OBE(L)} + \delta_{OBE(V)} + \delta_{OBE(T)}$
- δ_{TD} = component due to temperature differential (T_D) between superstructure and rail.

Figure 6-11: δ_{LF} definition

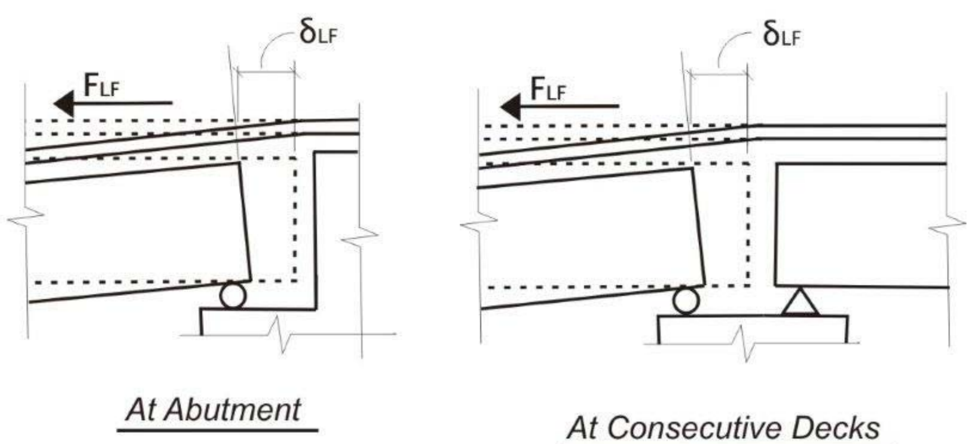


Figure 6-12: δ_{LLRM+I} definition

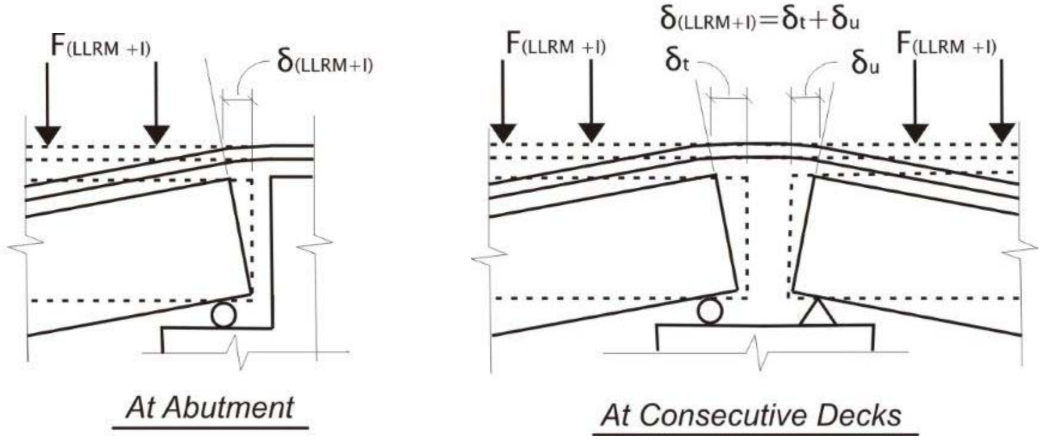
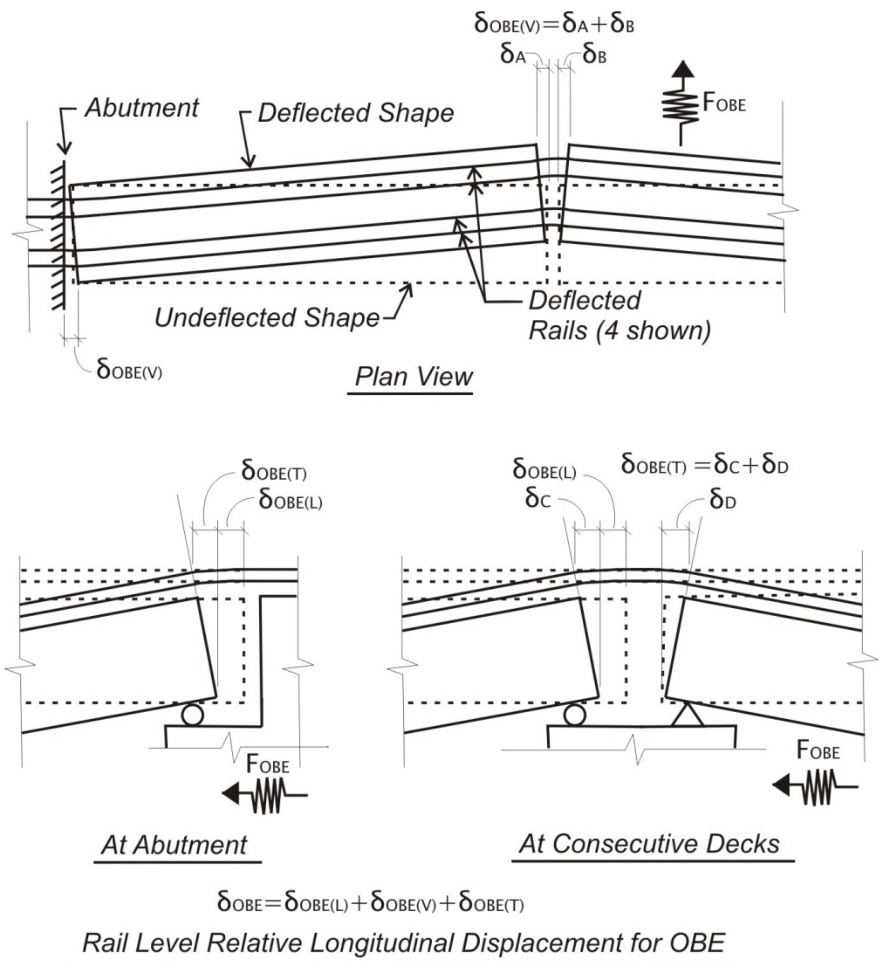


Figure 6-13: δ_{OBE} definition



The RLD at expansion joints measured relative to the original rail centroid locations (δ_L^{EXP}) shall not exceed the limits given in Table 6-10.

Note that in order to prevent having separate load cases for relative displacement and rail stress design, the expected temperature differential demands are added to the displacement limits. The temperature differential demands are dependent on the structural thermal unit (L_{TU}), which is defined as the point from fixed point of thermal expansion to the next adjacent fixed point of

thermal expansion. The maximum L_{TU} shall not exceed 330 feet without an approved design variance and special RSI analysis per Section 6.11.7.

Table 6-10: Relative Longitudinal Displacement at Expansion Joints Limits

Group	δ_L^{EXP} (inch)	
	Non-ballasted	Ballasted
4	$0.70 + \delta_{TD,Expected}$	$0.50 + \delta_{TD,Expected}$
5	$2.33 + 0.5\delta_{TD,Expected}$	$2.25 + 0.5\delta_{TD,Expected}$

where:

$\delta_{TD,Expected}$ = expected relative longitudinal displacement at the rail centroid due to T_D loading per Section 6.9.2.

For most structures, $\delta_{TD,Expected}$ can be approximated by:

$$\delta_{TD,Expected} = \alpha(\Delta T)L_{TU}$$

where:

α = coefficient of thermal expansion for the superstructure

ΔT = 40˚F temperature differential per Section 6.9.2. (ΔT always positive for calculation of $\delta_{TD,Expected}$)

L_{TU} = length of structural thermal unit for a given expansion joint.

For any structure where $\delta_{TD,Expected}$ cannot be approximated with the above equation, $\delta_{TD,Expected}$ shall be verifed by monitoring rail-structure interaction models subject to T_D loading. When a special rail-structure interaction analysis per Section 6.11.7 is required, a detailed temperature analysis shall be required to justify the determination of $\delta_{TD,Expected}$.

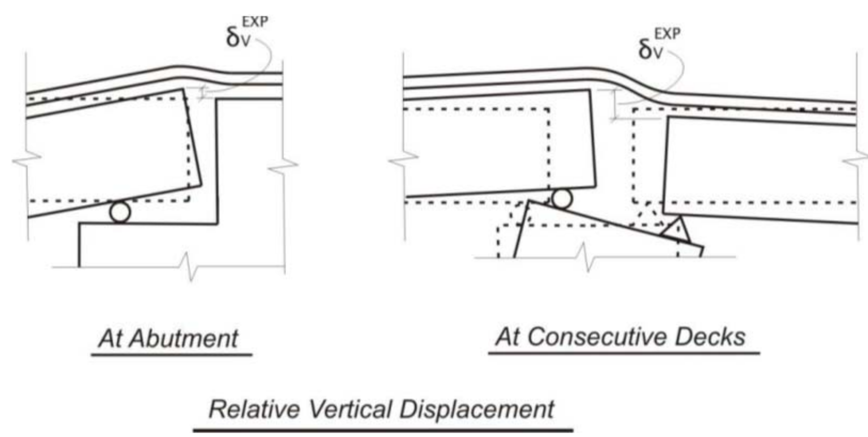
6.9.4 Relative Vertical Displacement at Expansion Joints

The relative vertical displacements (RVD) at structural expansion joints, δ_V^{EXP}, shall be limited in order to control rail bending stress..

The flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

As shown in Figure 6-14, structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

Figure 6-14: Relative Vertical Displacement at Expansion Joints



The RVD at expansion joints (δ_V^{EXP}) shall not exceed the limits given in Table 6-11.

Table 6-11: Relative Vertical Displacement at Expansion Joints Limits

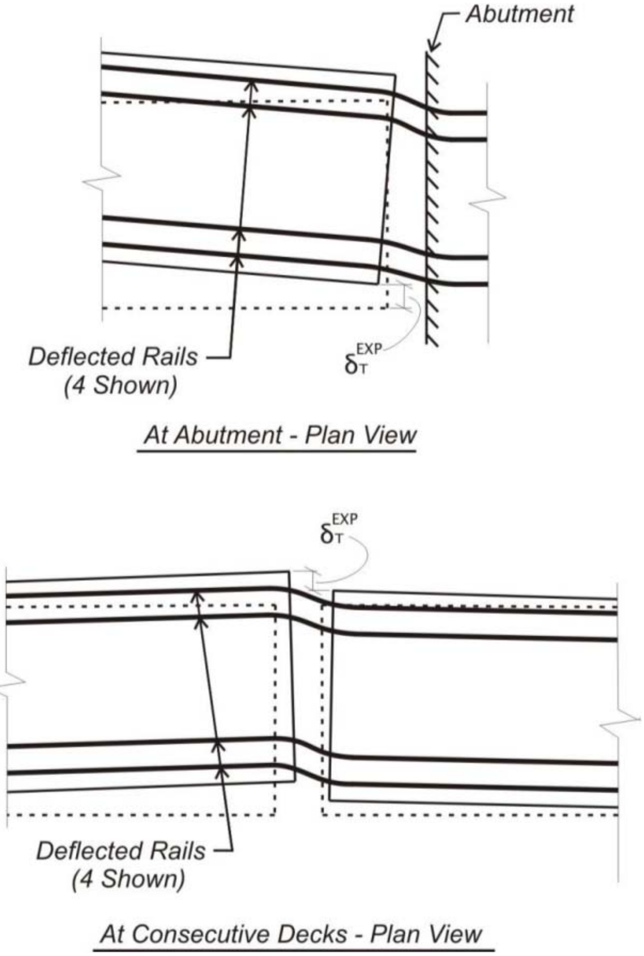
Group	δ_V^{EXP} (inch)	
	Non-ballasted Track	Ballasted Track
4	0.25	0.5
5	0.50	0.75

Refer to Section 6.8.9 for additional RVD limits for track serviceability analysis.

6.9.5 Relative Transverse Displacement at Expansion Joints

The relative transverse displacements (RTD) at structural expansion joints, δ_T^{EXP}, shall be limited in order to prevent excessive rail bending stress. As shown in Figure 6-15, structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

Figure 6-15: Relative Transverse Displacement at Expansion Joints



The RTD at expansion joints (δ_T^{EXP}) shall not exceed the limits given in Table 6-12.

Table 6-12: Relative Transverse Displacement at Expansion Joints Limits

Group	δ_T^{EXP} (inch)	
	Non-ballasted Track	Ballasted Track
4	0.08	0.16
5	0.16	0.24

6.9.6 Permissible Additional Axial Rail Stress Limits

Permissible additional axial rail stress limits were developed considering total allowable rail stresses minus bending stresses due to vertical wheel loads, relative displacements at structural expansion joints, and the initial axial rail stress due to rail temperature and preheat during rail installation (per TM 2.1.5: Track Design).

The permissible additional axial rail stress limits pertain to axial only rail stresses generated by RSI.

For rails on the TSI-critical structures and adjacent abutments or at-grade regions, the permissible additional axial rail stresses (σ_{rail}) shall be per Table 6-13.

Table 6-13: Permissible Additional Axial Rail Stress Limits

Group	Range of σ_{rail}	
	Non-ballasted Track	Ballasted Track
4	-14 ksi ≤ σ_{rail} ≤ +14 ksi	-12 ksi ≤ σ_{rail} ≤ +14 ksi
5	-23 ksi ≤ σ_{rail} ≤ +23 ksi	-21 ksi ≤ σ_{rail} ≤ +23 ksi

Note: Compression = Negative (-), Tension = Positive (+)

6.10 DYNAMIC STRUCTURAL ANALYSIS

6.10.1 General

Dynamic structural analysis using actual high-speed trains (LLV) is required in order to determine resonancy induced dynamic impact (I_{LLV}) effects, and limit vertical deck accelerations. Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loading.

For all dynamic structural analysis of high-speed train passage (LLV) the flexibility of the superstructure and substructure (i.e., bearings, shear keys, columns, and foundations) shall be considered.

To avoid over or underestimating the resonant speeds, two conditions must be investigated:

- Condition #1: lower bound estimate of stiffness and upper bound estimate of mass.
- Condition #2: upper bound estimate of stiffness and lower bound estimate of mass.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 6.11.

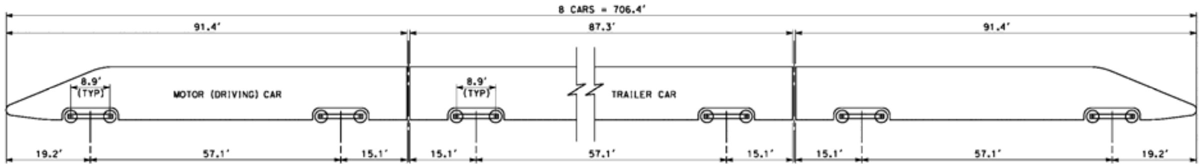
Refer to Section 6.5 to determine when dynamic structural analysis of high-speed train passage is required for preliminary design.

6.10.2 High Speed Train Loading (LLV)

Dynamic structural analysis of high-speed train passage shall consider representative trainsets (LLV), idealized as a series of moving vertical loads at specified axle and truck spacings. Modeling of the train suspension system shall not be required for dynamic structural analysis.

For preliminary design, when a limited dynamic analysis applies per Section 6.5.2, the single trainset shown in Figure 6-16 shall be investigated, subject to selected speeds given in Section 6.10.3.

Figure 6-16: Train Type 1 Loading Diagram



Maximum Axle Load = 18.7 tons   Train Weight (Empty) = 509 tons

6.10.3 Train Speeds

When a limited dynamic structural analysis applies per Section 6.5.2, one trainset shown in Figure 6-16 shall be investigated, subject to following selected speeds:

- The first two resonant speeds.
- Speeds at ±5 mph on each side of the fastest resonant speed.

Resonant Speeds

For simple spans, resonant speeds may be estimated by:

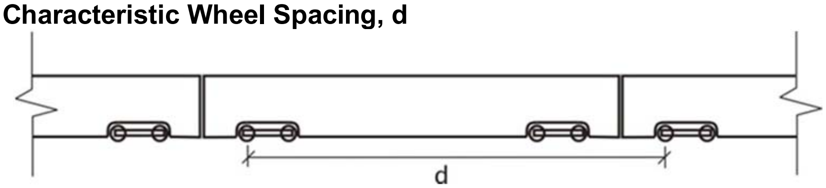
$V_i = n_o d/i$, where V_i = resonant speeds,

n_o = first natural frequency of vertical deflection

d = characteristic wheel spacing, refer to Figure 6-17

i = resonant mode numbers (e.g., 1, 2, 3, 4, …)

Figure 6-17: Characteristic Wheel Spacing, d



For structures not consisting of simple spans, resonant speeds shall be determined by the dynamic analysis model.

Cancellation Speeds

In addition to resonance, cancellation effects also contribute to the overall dynamic response of elevated structures. For simple spans, cancellation speeds may be estimated by the following:

$$V_i = \frac{2\eta_o L}{2i-1},$$ where V_i = cancellation speeds,

n_o = first natural frequency of vertical deflection

L = simple span length

i = cancellation mode numbers (e.g., 1, 2, 3, 4, …)

When L/d = 1.5, an optimal design condition exists for which the first mode of resonance aligns with the second mode of cancellation. In this condition, the primary dynamic residual response generated by repeated axle loads can be suppressed . Due to uncertainities associated with the service life of the structure, it may be unrealistic to design a given structure solely for a single characteristic wheel spacing. Nevertheless, optimal span lengths for potential trainsets shall be considered for design.

For non-simple span structures, the interaction between resonant and cancellation speeds may not be readily apparent and shall be investigated by a more detailed dynamic analysis.

6.10.4 Dynamic Vertical Impact Effects

For preliminary design, dynamic vertical impact effects need only be considered when a limited dynamic structural analysis applies per Section 6.5.2. The trainset in Figure 6-16 shall be investigated.

For the high-speed trainsets (LLV), the dynamic model shall be used to determine the dynamic impact effect (I_{LLV}) [2].

In order to determine (I_{LLV}), the maximum dynamic response value, ξ_{dyn}, shall be found for each structural response for single track loading (LLV) over the range of speeds given in Section 6.10.3.

Compared against the corresponding static response value, ξ_{stat}, the dynamic impact effect is as follows:

$$I_{LLV} = \max\left[\frac{\xi_{dyn}}{\xi_{stat}}\right]$$

6.10.5 Vertical Deck Acceleration

Vertical accelerations of TSI-critical structure decks are limited to avoid unsafe wheel-rail contact, and also to minimize passenger discomfort.

When evaluating vertical deck accelerations, an upper bound estimate of stiffness and lower bound estimate of mass, shall be investigated.

Vertical acceleration of TSI-critical structure decks shall be found for single track loading (LLV) over the range of train speeds given in Section 6.10.3. The vertical deck acceleration shall be monitored at the centerline of the loaded track.

The maximum vertical deck acceleration shall be limited to the following:

- +/- 16.1 ft/s^2 (0.50g) for non-ballasted track [3].
- +/- 16.1 ft/s^2 (0.35g) for ballasted track [3].

Note that this limit pertains to accelerations at the top of structural deck, not within the car body.

6.11 Modeling Requirements

6.11.1 General

The following modeling requirements for static and dynamic analysis of high-speed train TSI-critical structures are given for project-wide consistency.

6.11.2 Model Geometry and Boundary Conditions

The model shall represent the TSI-critical structure span lengths, vertical and horizontal geometries, column heights, mass and stiffness distribution, bearings, shear keys, column or abutment supports, and foundation conditions.

For isolated TSI-critical structures, with no adjacent structures, the model shall represent the entire structure including abutment support conditions.

For TSI-critical structures with repetitive simply supported spans, the model shall have a minimum of twenty (20) spans. Boundary conditions at the ends of the model shall represent the stiffness of any adjacent spans or frames.

For TSI-critical structures with repetitive continuous span frames (i.e., each frame consists of multiple spans with moment transfer between the deck and columns), the model shall have a

minimum of five (5) frames. Boundary conditions at the ends of the model shall represent the stiffness of adjacent spans or frames.

Soil springs at the foundations shall be developed based on information provided by the Project Geotechnical Design Report.

For modeling of earthen embankments or cuts at bridge approaches, refer to Section 6.11.9.

6.11.3 Model Stiffness

Structural elements shall be represented by the appropriate sectional properties and material properties.

For frequency and dynamic structural analysis, both upper and lower bound estimates of stiffness shall be considered.

For track serviceability and RSI analysis, a lower bound estimate of stiffness shall be considered.

For steel superstructure and steel column members, the following shall apply:

- Upper bound stiffness: full steel cross sectional properties, and expected material properties (larger than nominal specified per AASHTO LRFD BDS with California Amendments) shall be used.
- Lower bound stiffness: reduced steel cross sectional properties considering shear lag effects if necessary, and nominal material properties shall be used.

For reinforced, pre-stressed, and post-tensioned concrete superstructure members, the following shall apply:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used. Consideration shall be made for composite action of the superstructure with slab track, and barriers or derailment walls when determining upper bound bending inertias.
- Lower bound stiffness: effective bending inertia, I_{eff}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

For concrete column members, the following shall apply:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used.
- Lower bound stiffness: cracked bending inertia, I_{cr}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

As an alternative to using I_{cr} per CSDC, an effective bending inertia, I_{eff}, which considers the maximum moment demand, M_a, and the cracking moment, M_{cr}, may be used in accordance with AASHTO LRFD BDS with California Amendments. Also, an axially dependent moment-curvature representation of the column stiffness may be used.

6.11.4 Model Mass

For frequency analysis and dynamic analysis using actual high-speed trains, both upper and lower bound estimates of bridge mass shall be considered.

For track serviceability and RSI analysis, an upper bound estimate of bridge mass shall be considered.

For structural dead load (DC) mass, the material unit weights per TM 2.3.2: Structure Design Loads shall be used as the basis for design. For upper bound mass estimate, these unit weights shall be increased by a minimum of 5%. For lower bound mass estimate, these unit weights shall be reduced by a minimum of 5%.

For superimposed dead load (DW), upper and lower bound mass estimates shall be considered.

6.11.5 Model Damping

When performing OBE time history analyses for track serviceability and rail-structure interaction analysis, damping per TM 2.10.4: Seismic Design Criteria shall be used.

When performing dynamic analysis using actual high-speed trains, the peak structural response at resonant speed is highly dependent upon damping. The damping values in Table 6-14 shall be used [2].

Table 6-14: Damping Values for Dynamic Model

Bridge Type	Percent of Critical Damping
Steel and composite	0.5%
Pre-stressed, post-tensioned concrete	1.0%
Reinforced concrete	1.5%

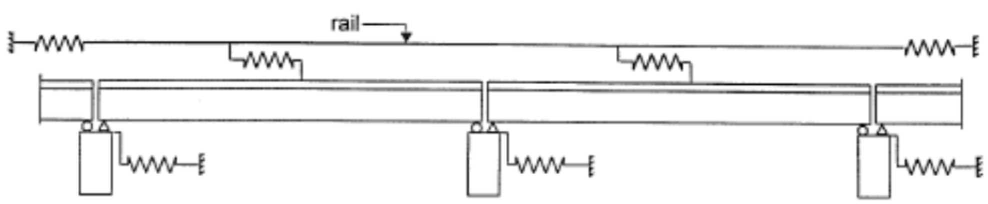
The damping may be increased for shorter spans (< 65 feet), see [2]. To justify use of increased damping, the Designer shall provide supporting evidence as part of the PDAP per Section 6.3.

When performing dynamic analysis using LLV, soil damping shall be considered in accordance with the Geotechnical Design Report.

6.11.6 Modeling of Rail-Structure Interaction

Longitudinal actions produce longitudinal forces in continuous rails. These forces are distributed to the TSI-critical structures in accordance with the relative stiffness of the track and fasteners, articulation of the structural system, and stiffness of the substructure, refer to Figure 6-18 for a schematic rail-structure interaction model.

Figure 6-18: Rail-Structure Interaction Model



Rail-structure interaction (RSI) may govern the following:

- Location and distance between bridge expansion joints
- Stiffness of the bridge superstructure
- Stiffness of the supporting columns and foundations

RSI shall be performed for all structures using either static or dynamic models. In addition, the model shall, at a minimum, include the axial stiffness of the rails appropriately located upon the superstructure, and longitudinal bi-linear coupling springs between the track and superstructure over the length of the model.

For purposes of this analysis, the continuous welded rail section shall be the EN60 rail per *EN13674-1, Railway applications – Track – Rail – Part 1: Vignole railway rails 46 kg/m and above*. Refer to Table 6-15 for calculated rail section properties to be used for analysis.

The use of the EN60 rail for analysis shall not be construed as a requirement for track design or track construction.

Table 6-15: EN 60 E1 Rail Section Properties

Property	Metric units (given)	US units (calculated)
Mass per meter:	60.21 kg/m	121.4 lb/yd
Cross-sectional area:	76.70 cm^2	11.89 in^2
Moment of inertia x-x axis:	3038.3 cm^4	73.00 in^4
Section modulus – Head:	333.6 cm^3	20.36 in^3
Section modulus – Base:	375.5 cm^3	22.91 in^3
Moment of inertia y-y axis:	512.3 cm^4	12.31 in^4
Section modulus y-y axis:	68.3 cm^3	4.17 in^3

The track type (non-ballasted or ballasted) and corresponding fasteners restraint shall be defined in the PDAP per Section 6.3.

Fastener restraint is nonlinear, allowing slippage of the rail relative to the track support structure. Bi-linear coupling springs shall represent non-ballasted track with direct fixation fasteners (refer to Figure 6-19) or ballasted track with concrete ties and elastic fasteners (refer to Figure 6-20) between the rails and superstructure on a per track (i.e., two rail) basis [2]. The non-ballasted relationship represents a pair of fasteners with 1.54 kip (6.85 kN) unloaded longitudinal restraint at 27-inch spacing. The ballasted relationship represents a pair of fasteners on a concrete tie with 1.54 kip (6.85 kN) unloaded longitudinal restraint at 27-inch tie spacing. In each case, the longitudinal restraint is 1.37 k (unloaded) per foot of track and 2.7 k (loaded) per foot of track. The yield displacement varies from 0.02” (non-ballasted) to 0.08” (ballasted).

Figure 6-19: Non-ballasted Track with Direct Fixation Fasteners: Bi-linear Coupling Springs

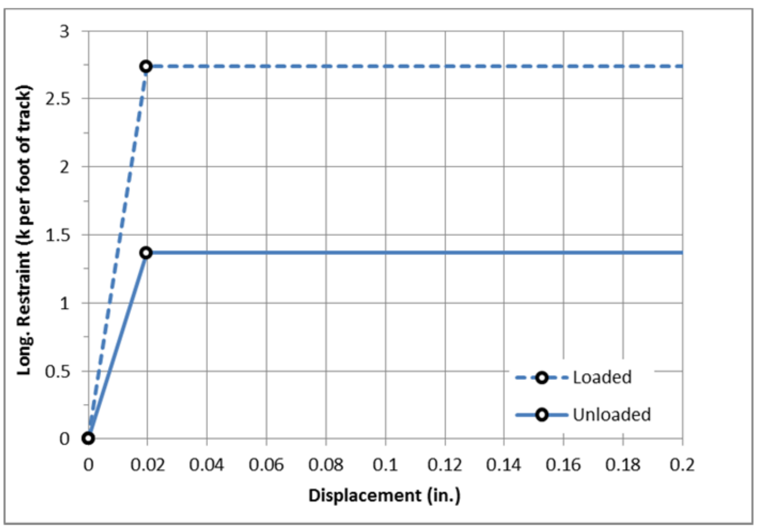
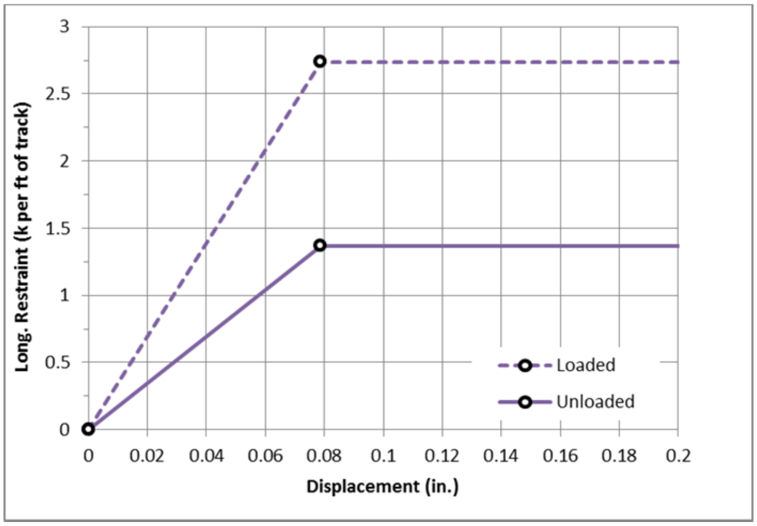


Figure 6-20: Ballasted Track with Concrete Ties and Elastic Fasteners: Bi-linear Coupling Springs



In practice, variations in fastener/tie spacing may be required to accommodate structural expansion joints, deck skew, or other geometric constraints.

Uniform longitudinal restraint shall be verified using the following uniformity criteria:

- Distributed longitudinal restraint calculated for fastener locations over any 10 foot length of track along the structure shall be within +/-20% of the assumed uniform bi-linear coupling relation.

For TSI-critical structures that meet the uniformity criteria, but are designed assuming longitudinal restraints which are not consistent with Figure 6-19 or Figure 6-20, the structure shall be considered to have a nonstandard fastener configuration (NSFC). These structures require an approved design variance and special RSI analysis per Section 6.11.7.

For TSI-critical structures that do not meet the uniformity criteria, the structure shall be considered to have a non-uniform fastener configuration (NUFC). These structures require an approved design variance and a special RSI analysis per Section 6.11.7.

The total number of longitudinal bi-linear coupling springs per each span shall not be less than ten (10) and the spacing between the springs shall not be more than 10 feet.

For vertical and lateral (i.e., transverse) stiffness of fasteners, defined as per foot of track (pair of rails) the following properties shall be used as applicable:

- Non-ballasted track:
    - Vertical stiffness: 4100 k/ft per foot of track
    - Lateral Stiffness: 420 k/ft per foot of track
- Ballasted track:
    - Vertical stiffness: 2100 k/ft per foot of track
    - Lateral Stiffness: 420 k/ft per foot of track

For purposes of evaluating TM 2.10.10 design criteria, constant vertical stiffness shall be used to model fastener compression and tension (uplift).

The assumed fastener stiffness relationships (longitudinal, vertical, and lateral) are to be used for the design of TSI-critical structures only. These relationships provided for RSI models are not to be used for track design. As a means to meet RSI criteria per Section 6.9, the Contractor may propose alternative track solutions (e.g., NSFC, NUFC, Rail Expansion Joints, etc.) through the design variance approval process. The design variance shall be supplemented with a special RSI analysis per Section 6.11.7.

6.11.7 Special Rail-Structure Interaction Analysis

RSI limits in Section 6.9 were developed considering typical fastener configurations on typical structures. For those systems that do not meet these assumptions, new limits shall be developed using a refined analysis.

A special RSI analysis shall be required for those structure and track designs requiring a design variance related to Section 6.9. Specific design variances requiring special RSI analysis include, but are not necessarily limited to: designs requiring nonstandard fastener configurations (NSFC), non-uniform fastener configurations (NUFC), structures with thermal units (L_{TU}) greater than 330 feet, rail expansion joints (REJs).

Examples of special analysis required may include, but are not limited to: development of new RSI limits, development of new analytical model elements, local rail stress modeling, site-specific temperature analysis, analysis of impacts to track maintenance.

6.11.8 Modeling of Rail-Structure Interaction at Model Boundaries

Where an abutment occurs at the ends of TSI-critical structures, the rails and bi-linear coupling springs shall be extended a distance of L_{ext} from the face of the abutment. At the model boundary (i.e., at L_{ext} from abutment), a horizontal boundary spring representing the rail/fastener system behavior shall be used. The boundary spring, which represents unloaded track, shall be elastic-perfectly plastic, with a elastic spring constant of k (in units of kips/feet) yielding at P_b (units in kips), which represents the maximum capacity of an infinite number of elastic fasteners.

The yielding of the boundary spring at P_b is a threshold value that shall be checked throughout the RSI analysis. If at any point during the analysis the boundary spring yields at force P_b, L_{ext} should be increased and the analysis should be repeated until elastic boundary spring behavior is verified.

The boundary spring behavior depends on the type of track adjacent to the analyzed structure. Values of k, P_b, and L_{ext} are given for non-ballasted and ballasted track types in Table 6-16. Note that the minimum recommended values of L_{ext} are dependent on the average span length of the TSI-critical structures (denoted L_{avg}):

$$L_{\text{avg}} = \frac{(L_1 + L_2 + \ldots + L_n)}{n} \text{ = the average span length}$$

Table 6-16: Minimum Recommended Track Extension and Boundary Spring Properties

Non-Ballasted Track (fasteners yield at 0.02 inches) with EN 60 E 1 rail			
Yield Load per foot of non-ballasted track	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.37 kips/ft of track [1.54 kips (6.85 kN) fasteners @ 27” o.c.]	23,800	39.7	$0.1L_{avg} + 325$
Ballasted Track (fasteners yield at 0.08 inches) with EN 60 E 1 rail			
Yield Load per foot of ballasted track	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.37 kips/ft of track [1.54 kips (6.85 kN) fasteners @ 27” o.c.]	11,900	79.5	$0.1L_{avg} + 300$

In the event that an additional bridge or other elevated structure is located within the L_{ext} model boundary distance from the face of an earthen abutment, the additional structure (including all loads and modeling requirements presented in this section) must also be included in the RSI analysis model.

The assumptions used to develop Table 6-16 were expected to apply to majority of TSI-critical structures, which are assumed to be in simply-supported configuration with uniform distribution of fasteners. Where a special rail-structure interaction analysis is required per Section 6.11.7, additional investigation shall be required to appropriately define the model boundary.

6.11.9 Modeling of Earthen Embankments or Cuts at Bridge Approaches

Where applicable under RSI Section 3.9.2 Group 4 and Group 5 load cases, the vertical and lateral stiffness of non-ballasted or ballasted track upon earthen embankments or cuts shall be determined to accurately predict relative displacements at abutment expansion joints, and rail stress at the abutment and at-grade regions.

The modeling of earthen embankments or cuts is not required for track serviceability (Section 6.8) or dynamic structural analysis (Section 6.10). However, if the embankment and rails are considered in these models, the vertical and lateral stiffness of non-ballasted or ballasted track upon earthen embankments or cuts shall be determined to accurately predict deformations and accelerations, where applicable.

Vertical stiffness of track upon earthen embankment or cuts shall be developed based upon the specific characteristics of the embankment, cut, or transition structure, as applicable. Guidelines can be found in TM 2.9.10: Geotechnical Analysis and Design Guidelines.

For lateral (i.e., longitudinal and transverse) stiffness of track upon earthen embankments or cuts, consideration of embankment flexibility, non-ballasted track or ballast tie embedment, passive pressure, and friction shall be made in accordance with the Geotechnical Design Report.

OBE ground motions shall be applied concurrently at structural foundations and earthen embankments or cuts to capture the effects between the vibrating structure and the relatively stationary track upon earthen embankment or cut. For tall embankments or specific soil types, lag times and/or amplification effects shall be considered for OBE ground motions in accordance with the Geotechnical Design Report.

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19 December 2013

PMT-CHSRA-03907

Frank Vacca
Chief Program Manager
California High-Speed Rail Authority
770 L Street, Suite 800
Sacramento, CA 95814

RE: Request for Authority Concurrence of TM 2.10.10 Track-Structure Interaction, R1

Mr. Vacca,

TM 2.10.10 Track-Structure Interaction, R1 is attached for your review and concurrence. This document provides the basis for structural design criteria based on system performance requirements of track safety, passenger comfort, and track maintenance. In addition, it presents preliminary design guidance for preliminary engineering and cost estimating. References are provided illustrating the use of similar criteria in other high-speed train systems. The following revisions have been made to reflect project decisions, Technical Advisory Panel review comments, and additional research:

- Criteria for structures supporting ballasted track were developed to supplement criteria for structures supporting non-ballasted track. Ballasted track-structure interaction criteria results in slightly stiffer structures to prevent ballast deconsolidation under service and operating basis earthquake (OBE) demands.
- TM 2.10.10 now considers EN 60 E1 (lower bound) and AREMA 141RE (upper bound) rail sections. Analysis requirements have been modified accordingly. The increased rail stresses caused by smaller rail sections has caused non-ballasted deformation limits to become slightly more stringent. For the purposes of rail-structure interaction analysis, the EN 60 E1 rail is required to be used.
- As a result of further research, the use of frequency limits as primary check for advanced analysis was de-emphasized for final design. For preliminary engineering, the frequency limits were revised to reflect speeds exceeding 220mph, based upon research developed by the Chinese. The upper bound frequency limits have been eliminated. The lower bound frequency threshold is set to be the sole basis for determining when a limited dynamic structural analysis is required.
- Miscellaneous revisions include train point load diagrams and train load case clarifications such as centrifugal force (CF) and accelerating and braking forces (LF). Group 1c loading has been revised to include explicit requirements for multiple track configurations.

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- Preliminary Design and Analysis Plan, which includes seismic and track-structure interaction (TSI) considerations, has been added to the preliminary design submittal requirements. This submittal will provide the Authority with the opportunity to review and comment on the preliminary modeling approach and assumptions.
- The requirement to evaluate vertical deck acceleration at the centerline of loaded track is added for the dynamic structural analysis.
- Additional discussion of Vehicle-Track-Structure Interaction (VTSI) analysis requirements for final design is provided.
- Further differentiation and clarifications are provided between preliminary versus final design requirements.
- Minor refinements including miscellaneous clarifications to terminology, acronyms, etc., have been made.

Many of the revisions were made due to the ongoing uncertainty regarding track and vehicle selection. It is understood that until some key parameters associated with track and vehicles are set, this Track-Structure Interaction criteria will remain as a living document and will be updated as required. If this meets with your requirements, please sign below acknowledging your concurrence for adoption and use on the program.

Regards,

James R. Van Epps

James R. Van Epps
Program Director

California High-Speed Rail Authority
Concurrence

Frank Vacca

Frank Vacca, Chief Program Manager

Date: 01/23/2014

Enclosure: TM 2.10.10 Track-Structure Interaction, R1

CALIFORNIA High-Speed Rail Authority

DATE: 12/20/2013

SIGNATURE/APPROVAL ROUTING SHEET

1244

DOCUMENT(S) INFORMATION

To: Jennifer Thommen

From: Kris Livingston

Subject: TM 2.10.10 Track Structure Interaction, Revision 1

Description of Enclosed Document(s):
The Technical Memo 2.10.10 Track Structure Interaction, Revision 1 is attached for your review and concurrence.

☐ Expedite Due Date:

REVIEWER INFORMATION

Reviewer #1 Name (Print): Jim Van Epps	Reviewer's Initial/Date: JRVE 12/20/13	Comments:
Reviewer #2 Name (Print): Frank Vacca	Reviewer's Initial/Date: FAV 1/23/2014	Comments:
Reviewer #3 Name (Print):	Reviewer's Initial/Date:	Comments:
Reviewer #4 Name (Print):	Reviewer's Initial/Date:	Comments:
Reviewer #5 Name (Print):	Reviewer's Initial/Date:	Comments:

☐ Approval/Signoff (initials)

☒ Signature

☒ Hand Carry or Call for Pick up
Name: Kris Livingston Ext.: 384-9515

☐ Information

☐ Do Not Release – Call When Signed

☐ Release When Signed

Executive Office Control No.:	Name of Contact Person:
	Phone Number:
	Office:
	Office Control No.: